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CXXXII.

RECONSTRUCTION AND

ENLARGEMENT OF CORK RUN TUNNEL

ON THE PITTSBURGH, CINCINNATI & ST. LOUIS RAILWAY. 1870-3.

A Paper by MAX J. BECKER, C. E., Member of the Society.

READ NOVEMBER 15TH, 1876.*

After crossing the Monongahela river at Pittsburgh, the line of the Pittsburgh, Cincinnati & St. Louis Railway follows the south bank of that stream to its junction with the Alleghany river, and continues along the left shore of the Ohio river for about one mile further to the mouth of Cork run, where it leaves the main valley and ascends, upon a 52 feet grade, to a tunnel in the summit range of the Ohio river hills. This tunnel is known as "Cork Run Tunnel," and is situated about 4 miles from the Pittsburgh depot. The construction of this railroad, then known as the Pittsburgh & Steubenville Railroad, was commenced as early as 1851, but in consequence of financial embarrassments was abandoned in 1856 in an unfinished condition.

At the time of the abandonment, the west approach of Cork Run Tunnel, 1 200 feet in length, was substantially finished. The west entrance was driven for a distance of 1 700 feet, lined with brick masonry and arched, and the heading was driven for about 100 feet further. The east approach, however, which is about 1 800 feet long and reaches a depth of 70 feet at the east portal, was only partly excavated and no

* Presented April 15th, 1876.

tunnelling had been done at the east end. All this work was constructed for a single track railway; the clear width of the tunnel between the side walls which supported the semi-circular arch was 13 feet at the springing line and 12 feet at grade; the clear height from top of rail to crown of arch was 16 feet 2 inches. The total length of the tunnel, as then contemplated, was 2 100 feet.

The summit of the ridge is about 800 feet from the west entrance and reaches an elevation of 195 feet above grade. A shaft had been sunk from a point 800 feet west of the east entrance, from which the work had been pushed westwardly to a junction with the west entrance and eastwardly to a point about 300 feet from the east entrance. (See longitudinal section of tunnel, Plate I.)

In 1862, a new organization was formed under the name of the "Western Transportation Company," which resumed work on the tunnel and completed it early in 1865, substantially upon the plan, for single track, adopted previous to the suspension, with side walls partly of brick and partly of rubble masonry and brick arch above.

Soon after the opening of the road in 1865, portions of the old walls, completed previous to the suspension, began to fail, owing in part to imperfect material, soft brick and inferior cement, but chiefly in consequence of bad workmanship. In many places, the excavation had not been made of sufficient width and height to admit of the full thickness of the masonry, and instead of 5 courses of brick, the arches frequently contained only 2 or 3, and the irregular rubble masonry of the side walls consisted in many places of a mere lining from 4 to 6 inches in thickness laid in inferior cement. When these yielded, the arch would follow. The packing above the arch had also been neglected, and it frequently happened that at points where the roof had been broken and timbering made necessary, large hollow spaces remained between the heading timbers and the arch; as these heading timbers decayed or broke, under the accumulating weight of overlying material, the entire mass would fall with great force upon the arch, crushing it without a moment's warning and seriously endangering and interrupting the traffic of the road.

Repairs became necessary within a year after the opening of the road in 1865, and were continued almost without interruption, until the final reconstruction of the entire tunnel for double track. These repairs were of necessity very expensive; the contracted space compelled the removal of the scaffoldings at the passage of every train and caused frequent inter-

ruptions to the work. When, after three years of almost continuous repairs, I measured the amount of actual work and counted up its cost, it appeared that a cubic yard of brick masonry, contained in these repairs, had cost the enormous sum of \$69, the labor of laying the same alone amounting to \$45. At this rate, the lining of the tunnel with masonry for single track, would have cost \$232.50 per lineal foot.

In the meantime, experiments had been made in two other tunnels upon the line of this road, which fully established the fact, that defective single track tunnels can be widened and reconstructed for double track, during continuance of the traffic, at considerably less expense than for single track. It was therefore decided in July, 1870, in the future, to replace all defective work in Cork Run Tunnel by widening and rearch-ing it for double track, and a contract was made with Owens & Morney (of Pittsburgh) for the immediate widening and reconstructing of 300 feet of the old tunnel at its east entrance and also for an extension of the tunnel 300 feet eastwardly into the approach, it having been ascertained that such would be cheaper and safer than the widening of the open approach, which at this point reached 70 feet in depth, through material liable to decompose, cause falls and thereby obstruct the track.

The sketch (Plate II.) shows the cross sections of both single and double track tunnel. The widening was done upon both sides equally, so as to make the centre line of the original single track the centre line between the two tracks when completed. Two footing courses of sandstone form the foundations of the side walls, the upper courses being checked in skewback form, for the reception of the brickwork above; the side walls are curved with a radius of 25 feet, so as to make the clear width at the true springing line of 25 feet; the top arch is a semicircle of $12\frac{1}{2}$ feet radius: the brick walls are 26 inches thick, all laid in mortar of hydraulic cement;* the façades are of cut sandstone, and the brick masonry of the arch consists of 6 courses, so arranged that whenever the outer ring had gained one course of brick, and coincided on a radial line with the inner ring, a few binder courses are introduced.

The solid rock, which reaches from the base of the tunnel to a height of about 21 feet, is overlaid near the crown of the arch, by a seam of fire-clay, 4 feet thick; above this appears another layer of rock about 2 feet thick, which is again overlaid by a heavy deposit of fire-clay, about 16 feet in thickness. The rock is of the bluish gray color common to the argillaceous stones in the coal formations of that locality; it is very

* Of R. H. Beeson's manufacture at Uniontown, Pa.

compact when in place, but liable to decompose after removal. The fire-clay is very difficult to work, on account of its toughness and the numerous irregular fissures and seams with which it is perforated. Wherever it was possible, we endeavored to preserve a portion of the solid rock as a roof support, but in many places this was found impracticable, either on account of the total disappearance of the rock below that elevation, or because the rock covering was of insufficient thickness to resist the pressure of the overlying masses of fire-clay. Great care had to be exercised wherever the headings had to be driven through this clay, and heavy timbering became necessary to support it, previous to and during the construction of the masonry. In nearly all cases, however, the struts supporting the head beams could be footed in the solid rock from 16 to 20 feet above grade and long posts were found unnecessary. When the solid rock above the crown of the arch was found of sufficient thickness to support the overlying deposits with safety, timbering was entirely dispensed with.

Before the contract with Owens & Morney for the 300 feet of reconstruction of old tunnel and the 300 feet of extension, had been entirely completed, it was decided to continue the work of widening, selecting the worst places as the first portions to be rebuilt. Soon afterwards, however, the widening of the entire tunnel, as part of the extension of double track from Pittsburgh westwardly, became a matter of immediate necessity.

In March, 1871, a new contract was concluded with Nugent & Douglass, for the widening of 1000 lineal feet of tunnel during the year 1871. To insure this rate of progress, it became necessary to commence operations at several points, so as to admit of the simultaneous advance of the mining, as well as of the arching, without delaying or interfering each with the other, and to prosecute the work by night and by day, uninterruptedly.

The usual mode of proceeding was as follows: a selection was made of the points at which the different sections of work were to be commenced; these selections were made with a view of pushing the work right and left at about equal rates of progress, and of uniting the sections at the two extremes simultaneously. At each of the selected points of attack, a manhole was first broken in the crown of the old arch, large enough to admit a miner with his tools; this miner would drive an entry, and with the assistance of additional men would push as rapidly as possible a heading for the enlarged section of the tunnel. The excavated

material was dropped through the manhole, upon flat cars, in waiting upon the track below. These headings were made in lengths varying from 16 to 32 feet, according to the condition of the roof and then extended sideways and downwards towards the springing lines of the arch, thus shaping the excavation to the proper cross section of the enlarged tunnel to receive the masonry. Where the heading was in fire-clay, or where only a thin seam of rock remained in the crown, too weak to support the pressure from above, timber struts were introduced, with their footings in recesses of the rock, as shown in Plate II. The timber struts, placed at intervals of from 3 to 5 feet apart, were covered with lagging and wedged to the roof.

In this manner the old arch answered well as a scaffolding for the miners employed in driving the headings. When a section of the heading for double track was completed as above described, the old arch beneath it was broken down and the material removed by railway. The old side walls were next taken down, and the sides of the tunnel were rapidly excavated to the full dimensions of the enlarged cross-section. The foundations of the new work were then laid, the side walls built and the arch of the section completed, while the miners pushed the heading and excavation and the removal of old masonry of the adjoining section in manner above described. In closing the arches of the last sections in each division, cast iron plates were used, which, when placed in position, formed a hollow key, through which the bricklayer could complete the arch and then drop a movable lid in the key, or slide a covering plate over the opening.

In this manner, the work of widening the tunnel—which was begun in July, 1870—was continued by day and by night, until its final completion in April, 1873. The regular traffic of the road, averaging 60 trains per day, was maintained during all this time with but few interruptions caused by accidents. The maintenance of this traffic necessarily interfered more or less with the blasting of the rock, the removal of the old masonry, the delivery of supplies, shifting of centres and scaffoldings, etc. The sulphurous smoke from the passing engines also proved a serious inconvenience to the workmen, especially the miners, while driving the headings above. The excavated material was transported by train a distance of about 3 miles and deposited in embankments by the contractor, without extra compensation; the old brick of good quality was cleaned and used again in the new work, the contractors being charged with the value.

The prices paid for this work were as follows :

1°. Contract with Owens & Morney for work done in 1870. Excavation of enlarged tunnel, averaging 10 cubic yards per lineal foot and including removal of old structure, \$50 per lineal foot ; brick masonry in tunnel, \$11, stone foundations, \$15, and brick masonry in extension of tunnel, \$10—each per cubic yard ; excavation of approach for extension, \$40 per lineal foot. Counting 4.582 cubic yards of brick-work and 0.63 cubic yards of stone foundations—per lineal foot—the cost of reconstructed double track tunnel under this contract amounts to \$109.85 per lineal foot. This however, does not include cement nor the expenses of the construction train, both being furnished by the railroad company.

2°. The contract price paid Hugen & Douglass for work done during 1871, was \$105 per lineal foot, exclusive of cement and construction train.

3°. The contract price paid John Douglass for work done in 1872, and until its completion in 1873, was \$120 per lineal foot of completed double track tunnel, exclusive of cement and train expenses.

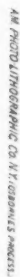
After the final completion of the work, it was found that the entire cost of the widening of the tunnel for double track, together with the 300 feet of extension and including cement, train expenses, superintendence, and all incidentals, amounted to very nearly \$150 per lineal foot.

The regular traffic of the road was interrupted, by falls from the roof, seven times during the reconstruction of this tunnel, causing detentions from 6 hours to 3 days. Five deaths of workmen occurred from various accidents.

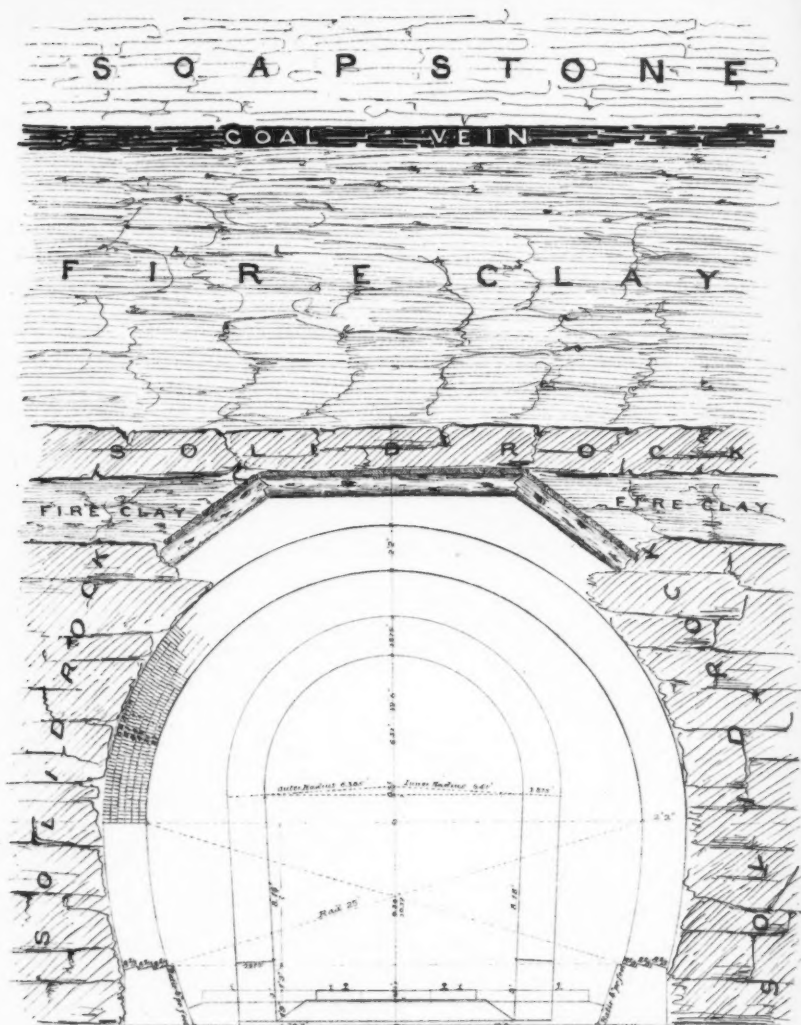
Within seven years from the opening of the line of the railway, it was thus found necessary to expend \$360 000 additional money for enlargement of this tunnel, after an outlay of not less than \$200 000 in its original construction, and about \$5 000 for subsequent repairs, making a total of \$565 000 ; while the building of the tunnel for double track in the first place could not have exceeded \$300 000, or less than its final widening for double track.

The instruction furnished by this experience, may perhaps, serve as a guide to other railway companies in shaping their policy of construction under similar circumstances. It proves, at least, that the postponement of permanent construction until after the opening of the road, which is so often practised under plea of economy, is not always the cheapest.

SCALE: VERTICAL, 75 FT. PER INCH
HORIZONTAL, 750' "







Areas per lineal foot Double Truck Tunnel
 Excavation 622.65 sq. ft.
 Brick Arch 123.33 "
 Stone Foundation 17.10 "

Areas per lineal foot Single Truck Tunnel
 Excavation 206.66 sq. ft.
 Brick Arch 45.11 sq. ft.
 " Walls 16.67 "
 Total Arch 75.76 "
 Stone Foundation 71.25 "

Plate II.
 CROSS SECTION OF CORK RUN TUNNEL.

AN PHOTO-LITHOGRAPHIC CO. N.Y. (GARDNER'S PROCESS)



CXXXIII.

NOTES ON THE

MASONRY OF THE EAST RIVER BRIDGE.

A Paper by FRANCIS COLLINGWOOD, C. E., Member of the Society.

PRESENTED NOVEMBER 1ST, 1876.

Having spent considerable time during the past winter in getting together the details of the masonry and attachments of the East River bridge, so far as completed, and in revising the estimates of the work so far as practicable, the writer desires to place on record the following details respecting the two towers and anchorages.

The figures referring to the towers will be given chiefly in connection with the Brooklyn tower and important differences only will be noted. The principal dimensions of these structures are as follows :

1° HEIGHTS.

BROOKLYN TOWER :

Bottom of foundation below mean high tide.....	44 feet 6 inches
Base of stone masonry.....	26 " 0 "
Depth of water in immediate front of tower.....	12 to 16 " 0 "
Height of roadway above mean high tide.....	119 " 3 "
" springing of arches above high tide.....	198 " 0 "
" " " roadway.....	79 " 3 "
" ridge of roof stone " mean high tide.....	271 " 6 "
" " " bottom of foundation.....	316 " 0 "

NEW YORK TOWER :

Bottom of foundation below mean high tide.....	78 " 0 "
Base of stone masonry " " ".....	46 " 6 "
Depth of water at immediate front of tower.....	34 " 0 "
Bottom of foundation to ridge of roof.....	349 " 6 "

In addition to all, there will be a balustrade around the towers on the cornice at the roadway and also at the edge of the roof slopes. This will increase the height to 276 feet above tide.

2° AREAS, OR HORIZONTAL SECTIONS.

At bottom of foundation (that is—bottom edge of caisson) :

Brooklyn tower is.....	102 × 168 feet = 17 136 square feet.
New York ".....	102 × 172 " = 17 544 "

At top of timber, the extreme measurements of the base of the masonry are : for

Brooklyn tower, 151 × 49 feet, with a solid section.....	= 8 542 "
New York " 77 × 157 " " ".....	= 9 115 "

At high water surface, the extreme measurements are :

Brooklyn tower, 57 × 141 feet, with a solid section.....	= 5 172 "
New York " 59 × 141 " " ".....	= 5 968 "

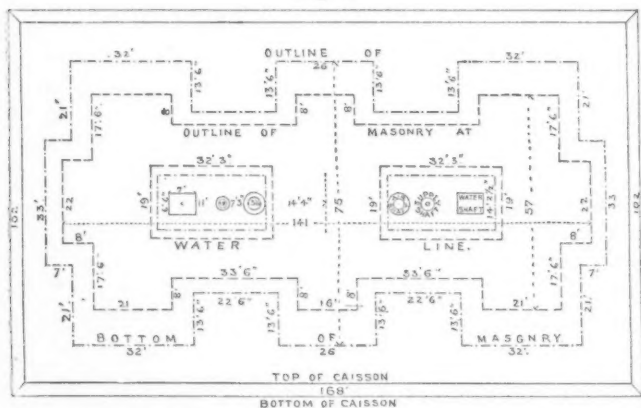
5°. PRESSURES.—From the foregoing we get, for the Brooklyn tower :

Pressure at bottom of foundation, about.....	5½	tons per square foot.
“ “ base of masonry (per square foot of bed) about. 9½	“	“ “ “
“ “ high tide (per square foot of bed).....	13	“ “ “
“ “ base of central shaft above roadway, final (pressure), about.....	26	“ “ “

The latter is the greatest pressure at any point in the tower masonry, being but 361 pounds per square inch, and it includes the pressure resulting from the two central cables. At the bottom of the foundation of the New York tower, the pressure is about 6½ tons per square foot of area, and at the base of the masonry, about 10½ tons per square foot of bed. The pressures at the bases of the masonry will be increased about 8 per cent. by the weight of the superstructure and load.

6°. DESCRIPTION OF TOWER MASONRY.—The general form of the tower masonry is shown in plan by the following. (Fig. 1.) It consists of three buttressed shafts, joined together so far as the roadway, by four connecting walls. At the course next the timber in the Brooklyn tower, these walls are 17 feet thick ; this thickness diminishes by offsets, until at high water and above, it is 10½ feet only.

Fig. 1.



Below high-water, the two well holes thus formed are filled with concrete and from high-water to the roadway, they are left open. Water, in considerable quantities,* collected in them during construction.

* This was the means of saving the life of a workman, who fell a clear descent of 105 feet. He struck upon an empty barrel floating in the water, crushed it and escaped with no injury beyond a few bruises.

Similar well holes or spaces were left from 2 feet above the arches to within $4\frac{1}{2}$ feet of the top of the tower, but each of them is divided by an interior connecting wall (parallel to the others), thus making four spaces of 4×33 feet section and 25 feet high.

The only other space in the masonry is a small vertical opening in one of the side shafts, 2 feet 5 inches \times 3 feet, starting just above the springing of the arch and connecting with one of the well holes above. By means of an iron ladder in these openings, and a trap through the roof courses, permanent access can be had to the roof.

Each arch has a span of 33 feet 9 inches. The arches are pointed and formed by the intersection of two arcs of circles described from centres in the springing plane, with radii of 45 feet $9\frac{1}{4}$ inches. The extrados has a radius of 48 feet 2 inches, with centres in the springing plane, so placed as to give a thickness of arch of 5 feet at the springing and of 4 feet at the point. The points of the arches are 114 feet 4 inches above the roadway.

The changes which have been noted in the dimensions of the horizontal sections are made by sloping offsets in the buttresses, each of 7 feet 6 inches rise. Two of these are below and two above the roadway, and they connect the several vertical sections mentioned.*

Leaving the subject of dimensions, the next thing in order is the character of the masonry. Throughout the work the specifications for stone have required, among other conditions, that the beds of all stones should be rough axed (or pointed), so as to allow of $\frac{1}{2}$ inch bed joints, with no pitch holes of more than 9 inches diameter, or $1\frac{1}{2}$ inches depth; and that the stone should be cut true to the rise. All vertical face joints from 4 feet below tide are $\frac{1}{2}$ inch; below this, only good quarry faces were required; this gave joints of 4 to 12 inches, and closer ones could be secured for the face, by selecting the stone. In dimension backing, the vertical joints were to be from 1 to 3 inches in width, and in all common backing, quarry faces were allowed.

Except where a face stone abutted against dimension backing, the rear face was left rough. Headers were allowed to be wider at the rear than the face, provided they were jointed back sufficiently for the adjacent stretchers. All face stone were paid for, according to the dimensions

* Through each of the spaces spanned by the arches there will be carried one track for cars propelled by steam, and two tracks for vehicles drawn by horses. In addition, there will be a foot track, which will be central in position and abut against the central shaft, passing around it both ways by tracks raised high enough to clear the cars; these tracks will be reached by flights of steps on each side of the shaft. This makes in all, 7 tracks on the roadway—4 for horses, 2 for steam cars, and 1 for foot way.

on the plan, deficiencies on each stone being charged for at contract price, and excess allowed for as backing. The rule was finally adopted and is now a part of every specification, that all stones of irregular shape (such as arch stones, cornice stones, &c.) shall be paid for, according to strict net measurement of their cubical contents; no constructive measurement being allowed. All bidders were notified to make their bids on this basis. The rises of the courses have been fixed in every case—the contractor bidding on specific rises. To accommodate the quarries, however, this has been varied between the limits of 20 and 30 inches; few being less than 27 inches, and the greater number being of 24 inches rise. The lengths of stretchers were fixed at from 6 to 15 feet, according to position, and the widths at from 3 to 5 feet. For ordinary stretchers, the contractor was allowed to vary in width within moderate limits, where it did not affect stability or plan.

Headers were made from 3 to 6 feet in width, the most common being from 3 to 4 feet, and none were allowed less than 6 feet long. The face stones ranged in cubical contents from $1\frac{1}{2}$ to 5 cubic yards each (the face key-stones measuring $5\frac{1}{2}$ yards each). The backing will average $1\frac{1}{2}$ cubic yards per piece in limestone, and $1\frac{1}{2}$ cubic yards in granite.

All but the cornice and offset stones from high water to the roadway, have a rock face of about 4 inches projection, and the arrises are pitched to a line. The corners of the buttresses have a vertical chisel draft of $1\frac{1}{2}$ inches width. The arrises of all the offset stones have the same draft, and the face between is pointed down to a $\frac{1}{2}$ inch projection. To heighten the contrast and produce the effect of horizontal bands, the granite in the offsets is of much lighter color than the rest of the stone. The faces of all the cornice stones are of six cent work (except on the sloping offsets as before described). Above the roadway, the face stones of the buttresses have the rock face projection reduced to about 3 inches; and a chisel draft is carried around the face of every stone. The pilasters facing the roadways are finished like the offset stones; and the intrados of the arches are smooth-pointed. The outer arch stones have a draft 3 inches wide, cut on the curved edge of the face and 2 inches wide on the other face edges; thus making a raised panel of $1\frac{1}{2}$ inches height, which is rough pointed between the drafts.

The voussoirs have a uniform thickness (length of curve on intrados) of 2 feet 3 inches. This gives a constantly diminishing thickness of the spandrel courses. The buttress and spandrel courses were, however, made to correspond in rise to the top of the thirteenth arch course (this having a rise on the spandrel of $23\frac{3}{4}$ inches). The eighteenth spandrel

course had a rise of only 18½ inches, and from the thirteenth to the eighteenth, the spandrel and buttress courses were unconformable. The bond was obtained by cutting down each and interlocking at the intersections, as best served to secure strong work. The study for this purpose was done by means of a model. The reason for not using thinner buttress courses and thus avoiding the necessity for such a construction, was, that the courses were too thin for the sizes of stone, required by the regular bond in the buttresses.

The general rule of the bond throughout the work, is two stretchers to one header.

7°. **MATERIALS USED IN MASONRY.**—The masonry of the towers below water is mostly of limestone, except the facing of the upper two courses, which is of granite. The backing above high water and below the roadway is mostly granite, all the remainder of the work being granite.

The anchorages were built entirely of limestone, with the exception of the corners, the front arches and the cornice. There were also about 650 cubic yards of heavy granite blocks in each anchorage, placed immediately over the anchor-plates, in order to secure a good hold upon the masonry above.

The limestone used has come from Kingston, Essex, Willsboro Point and Isle de La Motte, on Lake Champlain, and a small quantity from Canajoharie—all in New York. The granite of the towers has come from Deer Island, Fox Island, Mount Dessert, Blue Hill, Frankfort, Spruce Head, Green's Landing and Cape Ann—all in Maine; that for the anchorages came from Stony Creek, Conn.; Westerly, Rhode Island; Frankfort, Maine, and Charlotteburg, New Jersey.

The gravel used in concrete was beach gravel from various points on the north shore of Long Island. It contained a little sand, but was entirely free from dirt. The sand at first used was from Red Bank, off Staten Island; but the excavation for the Brooklyn anchorage furnished an abundant supply for all subsequent work. The cement used has been of the various brands known as Rosendale, no lime having been used at any time. This being a slow setting cement, there is a great advantage in using it where heavy blocks of stone are to be set and adjusted.

The ordinary proportions of sand, &c., used, have been by bulk—for mortar, one part cement and two sand; for concrete, one part cement, two parts sand, and four parts gravel; around the anchor bars and at some other points, the proportions were, one, two, and three.

No grouting has been permitted, except in rare cases where no other method was practicable; the general rule being, that all joints must be wide enough, at least for mortar; free use being made of "swords" and rammers, so as to insure perfect filling of joints.

In making up joints in the backing (wherever the spaces were wide enough to admit them,) broken stones of irregular sizes from a cubic foot down, were rammed into the concrete, until the spaces would receive no more. Great care was always taken to keep the work clean and to wet the faces of the stone. The face joints were dug out to a depth of $1\frac{1}{2}$ inches and pointed with pure cement mortar; a bead finish of about $\frac{1}{4}$ inch projection being put on with a tool. The pointing has never cracked, except when done too late in the season, or when the mortar was allowed to take its first set before using; re-tempered mortar is sure to crack.

In this connection it may seem proper to state that whenever necessary, the work has been carried on in freezing weather, and no bad results have been observed. The tops of the various pieces of work were always gone over carefully in the spring. The concrete which had been put in late would usually be found disintegrated to a depth of 1 to 4 inches; but below this it always was perfectly sound. The rule seemed to be that it was unsound only so far as it was exposed alternately to freezing and thawing, and wherever it had taken a set before freezing, and not been thawed out for some time, it was sound.

The total quantity of joints, both vertical and horizontal, for all the work done up to fall of 1875,* was as follows:—

Total masonry laid.....	120 235 cubic yards.
" stone used in masonry.....	100 015 " "
Excess of masonry over stone.....	20 220 " "
" " " in per centage of amount laid.....	16.8 " "

The excess is made up of mortar, concrete and broken stone; it is probable that the per centage is about 20 for the anchorages, and 15 for the towers; but the figures may be modified by the work of the past summer.

80. THRUST OF ARCHES.—Before construction, a careful analysis was made of the thrust, line of pressure, &c., in the arches under the various conditions of load to which they would be subjected. This showed that the centre line of pressures, resulting from the pier only, intersected the bases of the outer shafts at the roadway, at about one-third of their thickness

* Time has not permitted a revision for work done up to end of the present season.

from the outer faces; and that with the bridge completed, the intersection at this plane was close to the vertical lines through the centre of gravity of the shafts.

The deflection of the centre span of the cables is about 128 feet, and of the land spans about 187 feet, and the resultant of pressures from the cables falls in each pier slightly towards the river side of the centre line.

9°. IRONS INSERTED IN THE MASONRY.—To provide against possible changes of form, or accident during construction of the arches, the following precautions were taken :

At the top of the third voussoirs, 4 heavy irons were anchored into the masonry on each side of each arch, to which 3 inch-round iron rods spanning the arches were attached. Each rod was provided with a turn-buckle for tightening. Aside from serving to stiffen the arches, these rods served a very convenient purpose, as supports for scaffolds, while removing the centres and pointing the joints.

Permanent strengthening bars were inserted in both the first and second courses over the arches; there being in all 6 bars, $5 \times 1\frac{1}{4}$ inches, over each arch, anchored well into the shafts on either side.

Experience has shown the necessity of another precaution to obviate the evil effects of the unequal distribution of pressure over the base of the masonry. By simple inspection it will be seen that the pressure per square foot at the base of the connecting walls is less than half of that at the base of the shafts.* Hence there would be a tendency towards less compression and settlement under the connecting walls, and a consequent bulging upward at the roadway, causing vertical cracks in the connecting walls. This actually occurred to a limited extent in the Cincinnati bridge. To obviate this tendency, bars were inserted in top of the fourth, fifth and sixth courses below the roadway, in all 16 bars, $5 \times 1\frac{1}{4}$ inches, long enough to anchor into the shafts. In addition, the stones of the connecting wall for several courses were clamped together by $1\frac{1}{4}$ inch-round iron clamps.

Three sets of 2×10 inch-steel bars were inserted at each side of the pier, at the roadway, to serve as attachments for the under floor storm cables.

A set of flat iron bars were inserted at 20 feet below the roadway, as attaching points for holding down stays.

* By this is meant, the pressure at top of the timber or base of the masonry, from the connecting walls themselves, independent of what may be distributed from the shafts.

20 bars, $5 \times 1\frac{1}{2}$ inches, reaching entirely across the tower, were inserted in the top of the second course below the saddle plates; and in the course below this, 16 other bars, $5 \times 1\frac{1}{2}$ inches. The ends of these bars serve as attaching points for a portion of the long stays to the river and land spans of the roadway.

Small irons were inserted at frequent intervals to serve as attaching points for scaffolding, stairway, &c.

On each pier, on specially prepared beds, 4 saddle plates, each 8×16 feet, and weighing 11 tons each.

All irons were thoroughly galvanized before insertion. The saddles and plates were thoroughly coated with boiled linseed oil.

100. HISTORY.—The contract for the Brooklyn caisson was let October 25th, 1869, and work upon it began soon afterward. With the exception of surveys and office work, this was the first work done upon the bridge.

The diversity of standards and other inaccuracies, rendered it necessary to make a complete survey of the various streets, lots and buildings, on the line of the work; and it may be of interest to state that a line 6 000 feet long, and 300 to 500 feet wide, required four months' time for the completion of the survey.

The first ground was broken on the site of the Brooklyn pier, January 2d, 1870.

The Brooklyn caisson was launched March 19th, put in place May 2d, and the first stone set, June 15th, 1870; the interim between launching and stone setting being occupied, by putting on ten courses of timber, and various preparatory work. Work on the masonry stopped at an ultimate height of 6 inches above tide, December 10th, 1870.* By March 11th, 1871, the chambers of the caisson were filled with concrete, being at the average rate of about 50 yards per day.

It is worth while to note here that the balance of pressure against the caisson while lowering, was decidedly towards the river; and this resulted in a movement outward of the whole mass, of nearly 2 feet, during the descent. As this had been anticipated, and the base of the masonry made abundantly large, the position was readily corrected at the high-water surface.

March 11th, 1871, the laying of the masonry was again begun; closing December 29th, at 43 feet below the roadway, and 77 feet above tide. April 17th, 1872, work was again resumed; closing November 23d, at 24 feet above the roadway, and 144 feet above tide. March 20th, 1873, the

* The caisson did not reach its lowest point until December 22d, 1870.

work was again begun ; stopping October 30th, at the eighth arch course on 219 feet above tide. June 22d, 1874, work was begun again, and closed December 5th, at 267 feet above tide, or $4\frac{1}{2}$ feet below ridge of roof. Early in the spring of 1875, the saddle plates were set, and the saddles hoisted. All of the remaining stones were set but 78, which had to be omitted until the cables are made.

The total time occupied from the time of letting the first contract was five years and seven months. Reckoning from the bottom of foundation ; therefore the average height built per year, was 57 feet. This progress would have been considerably more rapid, had there been no delays from causes beyond the control of the engineering department.

The contract for the New York caisson was let September 6th, 1870 ; and it was launched May 8th, following. September 11th, 1871, it was put in place ; the 17 extra courses of timber (and concrete spaces) were completed between time of launching and October 31st, on which day the first stone was laid.

Stone laying, with the exception of a few very cold days, was continued all winter. This was necessary, in order to give sufficient weight to sink the caisson as the excavation proceeded. May 17th, 1872, the caisson reached its final resting place, and filling in began ; the masonry being at this date about 2 feet above tide. The chambers were all filled by July 22d. Although this time is shorter than that occupied in Brooklyn, the spaces to be filled were proportionally less. The masonry was stopped December 7th, 1872, at 60 feet above tide ; begun again April 1st, 1873, and stopped November 22d, at 6 feet above roadway, or 126 feet above tide ; begun again June 22d, 1874, and stopped December 12th, at 200 feet above tide ; begun again April 29th, 1875, and stopped November 27th, at 243 feet above tide ; begun again April 10th, 1876, and was finished, so far as possible, until after cable making, by the middle of July, or altogether in about five years and ten months.

Delays have attended this structure the same as the other. The final results show, that under all ordinary circumstances, about five years must be allowed for the erection of such a structure.

11°. METHOD OF ERECTION.—When the timber courses in the foundations were completed, 3 setting derricks were erected, one near the centre of each shaft. (See Plate VI, Figs. 26-35, 41-45.) These had masts of 56 feet height, with cross-booms 35 feet up, and strong back-stays to take the thrust. The booms were horizontal, with a buggy traversing their length, so that the derrick commanded the entire area of a circle of about 35 feet radius.

In Brooklyn, permanent hold backs (dead men) were used for guying to ; the system of guys being rectangular, and the three derricks being connected by sky-guys. This required a constant tightening of guys during the sinking of the caisson. In New York, the guys were attached to the top of heavy inclined struts 40 to 45 feet long, by eye-bolts with long screw threads for adjusting. The struts were double, with a cross-head of oak. On each side of the bolt for the sky-guy was another bolt for connecting the holding down rods ; these were finally anchored to heavy screw bolts in the timber below the masonry. The struts used at first had a spread of 6 feet at the base, and the rods, of 3 feet. As the work advanced and the rods were lengthened, this was found to lack in stability, and the spread was increased to 9 and 6 feet respectively.

In computing the dimensions of temporary iron work in detail, a load on wrought iron, of 10 tons was always assumed and maximum strains of 8 tons per square inch allowed ; the best Ulster iron being used, and for guys, steel or iron ropes.

For derrick falls, Manilla rope of best quality ($4\frac{1}{2}$ inches circumference, was always used ; and no rope allowed to be used after it became seriously chafed by wear.*

But two serious failures have occurred with these derricks. The first was by splitting of a cast iron rope socket on a back stay, when lifting a very heavy stone. On investigation it was found that the sockets were too light for the work ; and a heavier pattern was made which fully met the difficulty. The second failure was from an unsound weld in the eye, of a closed wrought iron wire rope socket. The open or two-jawed sockets have since been adopted, as being safer, and free from any such danger. This socket was also on a back stay.

With these derricks, a rise of masonry of 20 to 28 feet could be set without a change of position ; small holes being left in the masonry at the points occupied by the derricks, which could be filled after the derricks were raised. The cast-iron foot-step of the derricks had wrought iron rings in top by which they could be withdrawn, thus saving their renewal.

* No case of breakage of a fall ever occurred. Great care was always taken in leading a fall to prevent chafing. When possible, no leading block was put nearer to the drum than 20 feet, and the first blocks (which were usually permanent) were 18-inch iron sheaves lined with *metalline*, some of which has now been in constant use for eighteen months, with no perceptible wear. It has been found economical to turn the grooves of all sheaves, as a large saving in wear results from so doing.

At each successive raising, wrought iron brackets were inserted in the masonry, as supports for the feet of the derrick struts. By having the iron work prepared in advance, about one week was required for raising the 3 derricks on a pier, and about ten days for the 6 derricks on either anchorage. About 80 feet in height of the Brooklyn tower (above tide), and 120 feet of the New York tower, were thus laid.

As a hoist of 40 feet required 160 feet of rope to be coiled on the drum of an engine, it was found to be inconvenient to take more than this height for a single hoist; hence when this height was reached a tall derrick was erected at ground level, which raised all materials to a platform placed along the face of the tower. A railroad track on this, served for transporting the stone within reach of either of the setting derricks.

In New York, at 80 feet height, a second track was built, and the hoist taken to this, from the ground by a gaff attached to the face of the tower. To accommodate the increased length of rope, a smaller drum was made for the engine, so as to avoid the increased leverage caused by the extra turns.

The derricks used in unloading from the stone scows, and all those used at the stone yard, had the fall from the engine pass up through a hole in the centre of the footstep and spindle, and out along the inclined boom to a block at its upper end. (Plate VI, Figs. 37-39.)

The disadvantage of these derricks for stone setting, is that they only act on the circumference of a circle, and require the boom to be raised or lowered, to reach other points. Most of these were arranged with two fixed inclined struts from the head of mast, for supports.

There were several failures of the derricks with inclined struts—all resulting from the upward thrust from the strut, when a stone was suspended with boom towards the strut—*first*, from the breaking of the iron strap clasping the strut, at its head, (and in the end of which the derrick pin turns); and *second*, from the pin in head of mast being pulled out, (not being securely fastened). Another cause of failure was, the splitting of the foot-step; caused by the sides of the hole not being chamfered enough to allow lateral motion of the mast-head. In one case the pin at the mast-head, (2½ inches in diameter) broke square across, after about four years constant service.

All stones, were received, measured, assorted, and stored at the stone yard at Red Hook, where 15 derricks, and 3 steam engines were in constant use.

At each anchorage, 6 derricks similar to those used at first on the towers, were employed. The guying was in one system, requiring 7 sky-guys and 10 inclined guys, the load assumed being 8 tons, on each of 2 derricks hanging in the same direction; this load was very rarely if ever reached.

From the heights previously named, the towers were completed by the use of balance setting derricks, (Plate V.) and a travelling crane; the stone being hoisted to them by a 25 horse-power (nominal) engine, link motion, geared to 2 heavy drums, (Plate III, Figs. 6-9), carrying wire rope, of $1\frac{1}{2}$ inch diameter. These hoisted the stone to, and through, an elevated platform, (Plate III, Figs. 2, 3, 4, 10), supported by scaffolding, resting on the ground, and anchored to the masonry at about every 12 feet of height. The verticals of this scaffolding were compound beams each made up of 6 planks, and a center piece, all short; the planks broke joints regularly, were securely spiked, and through screw bolts placed at intervals. Each rope passed over a large wood lined sheave 4 feet in diameter, which was supported by a heavy oak frame securely guyed to the masonry, irons being inserted for the purpose. (Plates III, IV.)

The vertical support of each frame was a compound beam of 8 planks and a centre piece. It was securely strutted and anchored to the masonry and the elevated platform. The platform and frames were raised ordinarily for every 8 feet rise of masonry. The drums were disconnected for this purpose, and enough rope unrove to allow of the increased height.

A wire rope backing line joined the hooks of the two hoisting ropes, so that as one was raised the other was hauled down by the line. As only one gang of masons worked on the tower, this was found to be all that was required, as the hoisting could be done faster than the setting.

The average speed of hoisting was from 1 to 2 feet per second. A greater speed was not desirable, on account of the difficulty in properly handling the tag line.

Owing to the great length of the inclined portion of the ropes, there was a tendency in them to take up a vibratory motion, synchronous with the strokes of the engine, and the engineers were always instructed to change the engine speed occasionally, to arrest this action.

A double engine with cranks at right angles, would, on several accounts, have been preferable to the one adopted, chiefly for greater smoothness of action in lowering. The jerks accompanying this, particularly when with a heavy load, brought severe strains on all the

attachments ; fortunately, the occasions when such loads were lowered were very rare.

The balance derricks on the towers had arms of equal length, and the balance weights used were sufficient to balance safely a load of 6 tons. When stones heavier than this were being handled, the balance end was chained down to stones already set, until the stone was run in far enough to be balanced by the weight. Struts under the boom were at times used in setting a heavy stone at the end of the boom.

It was found most convenient to set a portion of the stones at the points midway between the derricks, by using a heavy oak beam suspended between the derricks, the stone being suspended and set from the centre of the beam.

On the Brooklyn tower these derricks were worked by small steam engines placed on them, and traveling around with them. The steam was brought up from the yard below in iron pipes, and carried to the engines in rubber hose.

The traveling cranes were used only on the central shafts, and when interfered with by the arches they were replaced by balance derricks. The cranes traversed in two directions, so as to reach every point underneath, and they were supported by scaffolding similar to that for the elevated platforms. They are safer than the balance derricks, and more expeditious in use.

The engines used on the work, aside from those spoken of, were of two kinds. In the first, an end V friction on one end of the drum was used for throwing in gear, in place of a clutch; and the lowering was done by a strap brake at the other end of the drum. These work with entire safety, but the descent of a stone in setting could not be governed so well as with the other style, which had a clutch on the driving shaft for throwing in gear, and a strap brake on the drum for holding the load. Most of these had two drums, each worked by a cylinder 8 inches in diameter and 10 inches stroke.*

For all ordinary weights with fourfold purchase, 85 pounds of steam were sufficient ; but for loads of 7 to 9 tons, steam was frequently carried at 100 pounds ; particularly when the hoist was up to 80 feet. The lowering and setting (the second type of engine was used at the anchorage for this purpose) with these was done by shutting off steam, and opening to a greater or less degree a small escape cock, connected with

* Most of the engines of this class in use on the work were made by Louis Osborne, South Boston, Mass.

the valve chest. With the descending load, the piston then acted as an air pump, and the speed of descent depended entirely upon the quantity of air allowed to escape.

12°. ARRANGEMENT OF WORKS.—From the very outset on this work, the fact that steam power is cheaper than hand labor has been kept steadily in view. At the stone yard, the 15 derricks mentioned were so arranged that all could be worked by steam, and a series of narrow gauge tracks were laid so that the stone could be brought to them on small stone cars with the least possible labor.

At the towers and anchorages, (Plate III), the permanent tracks were of ordinary gauge, 4 feet 8½ inches, and turn-tables and switches put in where required for convenient working. The advantages of the wider gauge were, greater freedom from danger of upsetting a car with a heavy load, and also in Brooklyn, the saving made by using the street car tracks for the distance of a block. Until the masonry of the New York tower reached high water, there were two tracks extended directly over the work to about the middle points between the derricks. These were suspended on cantilevers, so as to be entirely free from the work below and proved a great convenience. The tracks to the anchorages were made double (except in Water street, New York, which was too narrow), and curves were put in, wherever observation showed that by so doing any obstruction of frequent occurrence could be avoided, such as trucks unloading at a bonded warehouse. On the New York side, the anchor plates, measuring 16×17½ feet and weighing 23 tons, were transported from the dock to the anchorage over these tracks. Each plate was suspended between two cars, on a simple frame work of timber about 30 feet long; the cars acting as bogies. The curves in the track, one of which was of 34 feet radius, were greased, and with two teams of horses, the plates were each hauled to the anchorage 1 000 feet distant, in from 1½ to 2 minutes.

13°. ACCIDENTS*.—As is sure to be the case, these always happen where least expected, and at some weak point which has been overlooked. A system of daily inspection of derricks was instituted at the start and kept up to the end of the work. This was entrusted to the foreman in charge of riggers, a man of large experience. Aside from failures already mentioned, there were several cases of straightening of hooks of blocks under heavy loads. These were in every case, only after long,

* The term is used without regard to the injury done (it being in many cases very slight), the object being to point out the sources of danger on such work.

continued use, and evidently after the iron had been repeatedly strained beyond the elastic limit. In one case, with fortunate escape from serious consequences, a leading block on a derrick split. In another case the heel of an unloading derrick—frame-work and all—slipped on its bed. Several serious accidents, all the result of carelessness of those injured, occurred to men guiding falls on engine drums. One or more occurred from defective strap brakes; the woods having worn so as to prevent a proper tightening. One occurred from the careening of a balance derrick, owing to the improper removal of the blocking underneath; and one or two from overbalancing. It should be stated here that these derricks were raised 4 feet at a time, and supported on blocking, which was removed piece by piece as stone setting required.

The limestone first received was lewised with round lewises. These were found to be unsafe, but not until one or two serious accidents had resulted, by stone so lewised falling when swinging over the work. In all subsequent contracts, it was specified that flat lewises only should be used. (Plate VI, Fig. 40). The size adopted was such that the portion entering the stone had a united area for the 3 parts, of about 4 inches bottom width, 3 inches top width, and entering 4 inches into the stone, with a thickness of $\frac{3}{4}$ inch. These, when properly driven, never pulled out, with any hoist up to 80 feet. But when very heavy pieces of granite were taken to the upper portion of the tower, the surging would sometimes crack out a flake of stone, 1 to 2 feet wide. But one stone actually fell from this cause (from a height of 200 feet), but in several cases the flake came out, on removing the lewis. For this reason, a still larger lewis was used afterward on the heavier stones of from 9 to 11 tons weight.

Most of the serious accidents to men occurred from falling. One fell from a gang-plank by allowing his wheelbarrow to pull him off, and several lives of men working on the towers were lost by a mis-step. The force of the wind near the edge of the work was often very great, and, coming as it did, in sudden puffs, it became a source of great danger.

14°. SIGNALS.—At such a height as that of the towers, it was often difficult to make the men who were below understand signals from those above. This became a source of real danger in hoisting heavy stone, as the top of the tower was frequently enveloped in mist, so as to almost hide it from the engineer. The wind and other noises would often drown the voice, and at times a sharp whistle was heard but faintly. Bell wires were tried, but were so liable to become deranged that they were sometimes sources of danger. By great watchfulness, the cases of overwind-

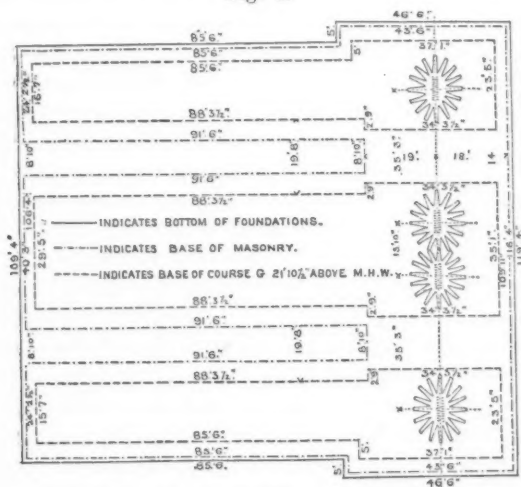
ing were rare, and no serious results followed. The safest signal was a flag, held well out, and the motions positively made, so as to be unmistakable if seen at all. A whistle was also used as an additional safeguard.

15°. ANCHORAGES.—These rest on timber foundations, with the spaces between sticks, 2 to 5 inches, and filled with concrete. The extreme dimensions of the Brooklyn anchorage foundation are, 119 feet 4 inches by 132 feet. It is 4 feet deep, its base reaching to tide level, and the whole being constantly wet with water in the sand. The excavation was from 20 to 25 feet deep, and the foundation rests on a uniform bottom of fine sand.

The New York anchorage has a similar foundation, 119 feet 4 inches wide, but extended at the front so as to be 138 feet long. This change was made on account of the character of the bottom across the front edge. The ground here had been filled in, and the excavation was continued until a uniform bottom of clean sand and gravel was reached over the whole surface. The foundation is all below tide, and has a depth of 4 to 7 feet.

The masonry of the Brooklyn anchorage, therefore, starts at 4 feet above tide, and that in New York at high tide level. The anchor plates in Brooklyn have their upper surface at 8 feet, and in New York, at 6 feet above tide. The general plan of the anchorages is here shown.

Fig. 2.



The walls, both outside and in the main tunnels, have a batter of $\frac{1}{4}$ inch per foot rise. The exterior measurements of the cornice are 124

feet, by 111 feet 8 inches at rear, and 101 feet 8 inches at front. The main tunnels are arched over by semicircular arches of 23 feet span, springing at 62 to 66 feet above tide. The rear tunnels have vertical walls, and are arched by semicircular arches of 14 feet span. These were not a part of the original plan, but were inserted to give means of communication from front to rear.

For 29 feet above tide in Brooklyn, and 22 feet in New York, the stones, except over the anchor plates, are all limestone, with rock face pitched to a line on the arrises. At these heights there is a 10 inch offset carried around all the faces except those of the rear tunnels, and above this the corner stones at each exterior angle are of granite. The corner stones have a bold chamfer, 4 inches broad, cut entirely around each face, except at the projecting corner. The faces between the chamfers have a draft $1\frac{1}{2}$ inches wide cut around each, and the surfaces between, pointed to $\frac{1}{2}$ inch projection. The limestone has the same finish throughout, as at first described. The cornices are of granite, corresponding in detail to those of the towers.

The New York anchorage contains 28 803, and the Brooklyn anchorage 27 113 cubic yards of masonry.

The anchor bars start from each plate in double sets, one curving over the other. They are vertical for about 25 feet, and then curve about 90° , so that the radius of a circle through the lower pins is 49 feet 6 inches. From this point, they extend to within 25 feet from the front edge of the masonry, where the cables are attached. The links of the first three sets have a section, 7×3 inches; the next three, 8×3 inches, and the next three, 9×3 inches. The tenth set is double in number, and each, $1\frac{1}{2} \times 9$ inches. The total weight in each anchorage is about 1 000 000 pounds. At each knuckle of the chains a large piece of granite is set, with a heavy cast iron plate inserted, as a bearing for the heads of the links.

Aside from these bars, there are heavy bars inserted for attaching the cradle and foot-bridge cables, for attaching the wind guys, and minor irons for temporary work.

The work of excavating for the Brooklyn anchorage began February 15th, 1873. The foundation was completed, and the first stone laid, June 26th. Work stopped November 29th, at 27 feet above tide. The approximate cost of the season's work, for 8 334 cubic yards of masonry laid, was \$18 per cubic yard, of which \$13.34 were for stone.

Work began again June 11th, 1874, and continued until December 29th, reaching 61 feet above tide; it was again resumed April 6th, 1875,

and was completed as far as practicable till after cable making, October 1st, 1875, a total of 24 132 cubic yards having been laid.

Work on the New York anchorage began about May 1st, 1875, and the first stone was set August 5th; work was closed December 11th, 1875, 15 067 cubic yards of masonry having been laid during the season, at an approximate cost of \$14.50 per yard; of which \$9.50 were for stone; work began again April 10th, 1876, and the masonry was completed ready for cable making August 31st.

Estimates have not been made up, for this season's work. The increased cost in New York for labor, etc., is due to the greater delays in handling material and to pressing the work, two gangs of masons being employed during the day and one at night, instead of one day gang.

Approximate estimates of the cost of the Brooklyn tower, as it stands, show that the masonry for labor and contingencies (or everything but materials used in masonry), cost per cubic yard, \$7.84. This includes labor, foremen, machinists, watchmen, scaffolding, wear and tear, rent of stone yard, towing scows, coal, &c., and may be subdivided as follows:

Top of caisson to high tide, per cubic yard.....	\$4.96
High tide to roadway, about " "	6.36
Roadway to springing " "	9.70
Springing to top, " "	12.60

Of this, the cost on the stone from the time it was laid alongside the dock, at Red Hook, until it was laid alongside at the pier, averaged \$1.10 per yard. If we deduct this, from the cost of each portion as previously given, we get the relative costs (independent of cost of stone) of the various portions about as the following numbers:

Below tide.....	13
Tide to roadway.....	18
Roadway to springing.....	29
Springing to top.....	39

In other words, the first 80 feet above the roadway cost about one and one-half times as much per yard for labor and contingencies as the 120 feet from high tide to roadway, and the 72 feet above springing about twice as much.

The average cost of the stone used in the Brooklyn tower, delivered at Red Hook, was about \$21 per cubic yard, varying all the way from \$15 to \$83 per cubic yard.

Excavation in the Brooklyn caisson, cost for labor only, including the men on top, about \$5.25 per cubic yard. Running the 6 air compressors added to this, \$3.60 per hour, or about 47 cents per yard; lights added, \$0.56 more; and these with other contingencies nearly equalled the cost

of labor. The great cost was due to the excessive hardness of the material over much of the surface; the caisson finally resting over nearly its whole extent on a mass of boulders, or hard pan.

The concrete in the caisson cost about \$15.50 per cubic yard for every expense. The caisson and filling together aggregated 16 898 cubic yards; and the approximate cost per yard for every expense was \$20.71. This was less than the cost of masonry laid in the open air.

The labor of making these estimates is very great; and it has not been done for the New York tower.

16°. SETTLEMENT OF MASONRY.—Bringing the account of the work up to the latest date, it is sufficient to say, in closing, that the settlement of the Brooklyn tower, at the time of completing the masonry (measured from marks at all salient angles, which were made immediately after the work reached high water), averaged 0.101 feet, the extremes being 0.08 and 0.102 feet. The average for the New York tower was $1\frac{1}{2}$ inches, with a still closer correspondence, but the figures are not at hand.

The settlement of the New York anchorage from the time of reaching 22 feet above tide when the levels of all salient angles were referred to a permanent bench mark, was $\frac{1}{2}$ inch across the front, and $1\frac{1}{2}$ inches across the rear. The difference is no doubt due to the greater proportional weight at the rear. The figures for the Brooklyn anchorage are about the same, but are not at hand.*

17°. DESCRIPTION OF PLATES.

PLATE III. Fig. 1.—Plan of tracks for handling stone at base of tower.

“ 2.—Plan of track and hoisting frame at top of tower.

“ 3.—End elevation of track and supporting frame at top of tower.

Fig. 4.—Part elevation of same.

“ 5.—Details of sheave for backing rope, at base of tower.

“ 6.—Drum connected with engine, at base of tower.

“ 7.—Longitudinal section of same.

“ 8.—Cross section of same.

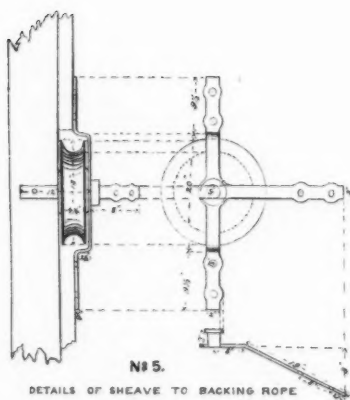
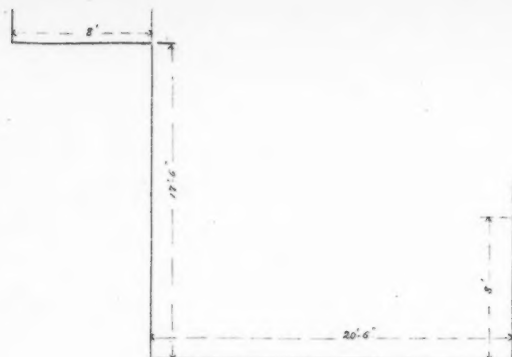
“ 9.—Enlarged section of same.

“ 10.—Car at top of tower.

“ 11.—Details of connection of backing and hoisting rope, and picking-up hook.

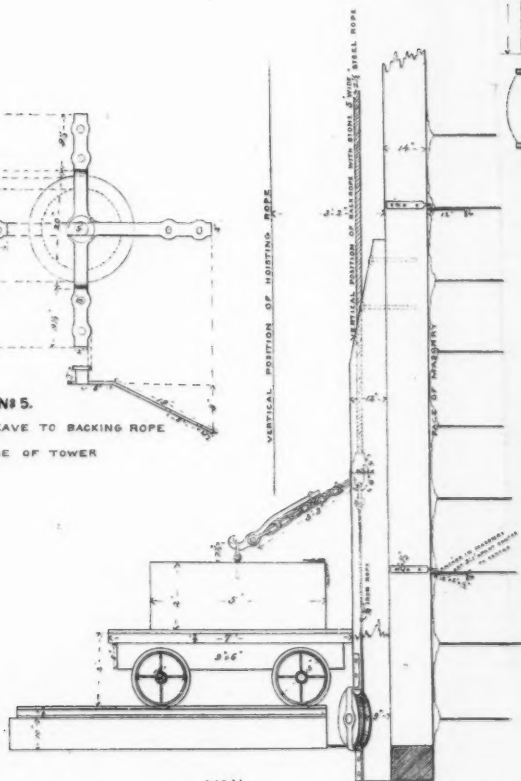
* Photographs accompanying this paper, showing the Brooklyn tower and the New York anchorage in perspective, are deposited at the rooms of the Society.

Plate III.



Nº 5.

DETAILS OF SHEAVE TO BACKING ROPE
AT BASE OF TOWER

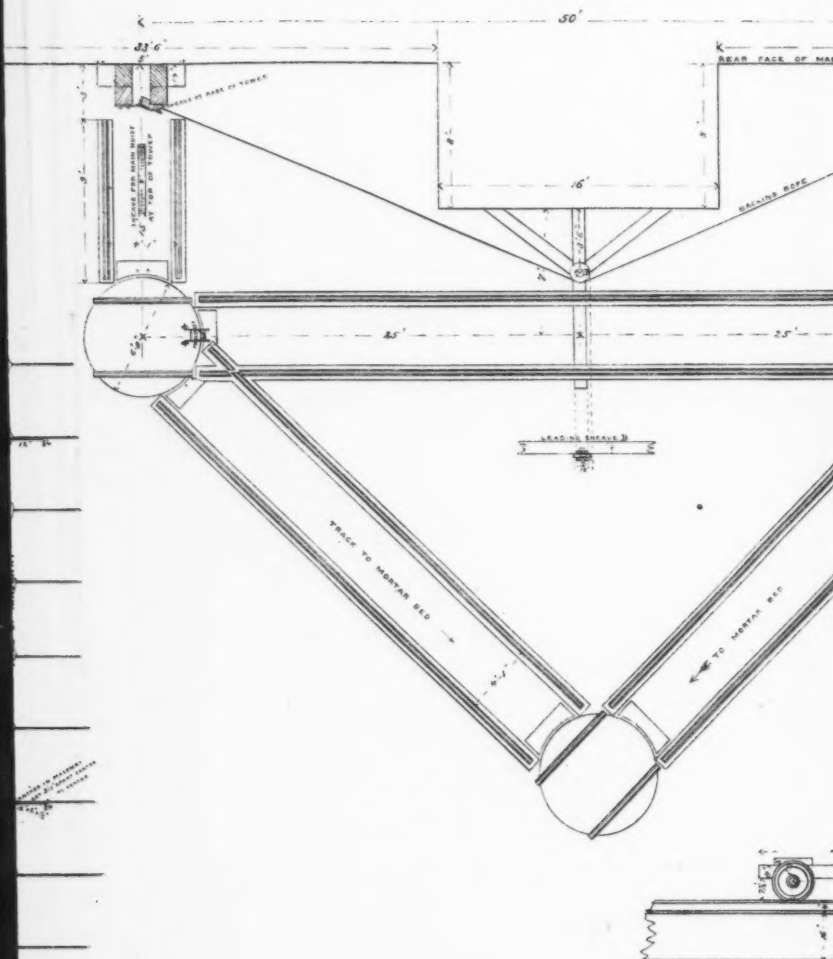


Nº 11.

DETAILS OF CONNECTION OF BACKING AND HOISTING ROPE
AND OF PICKING UP HOOK

N^o 1.

PLAN OF TRACKS FOR HANDLING STONES AT BASE
OF TOWER

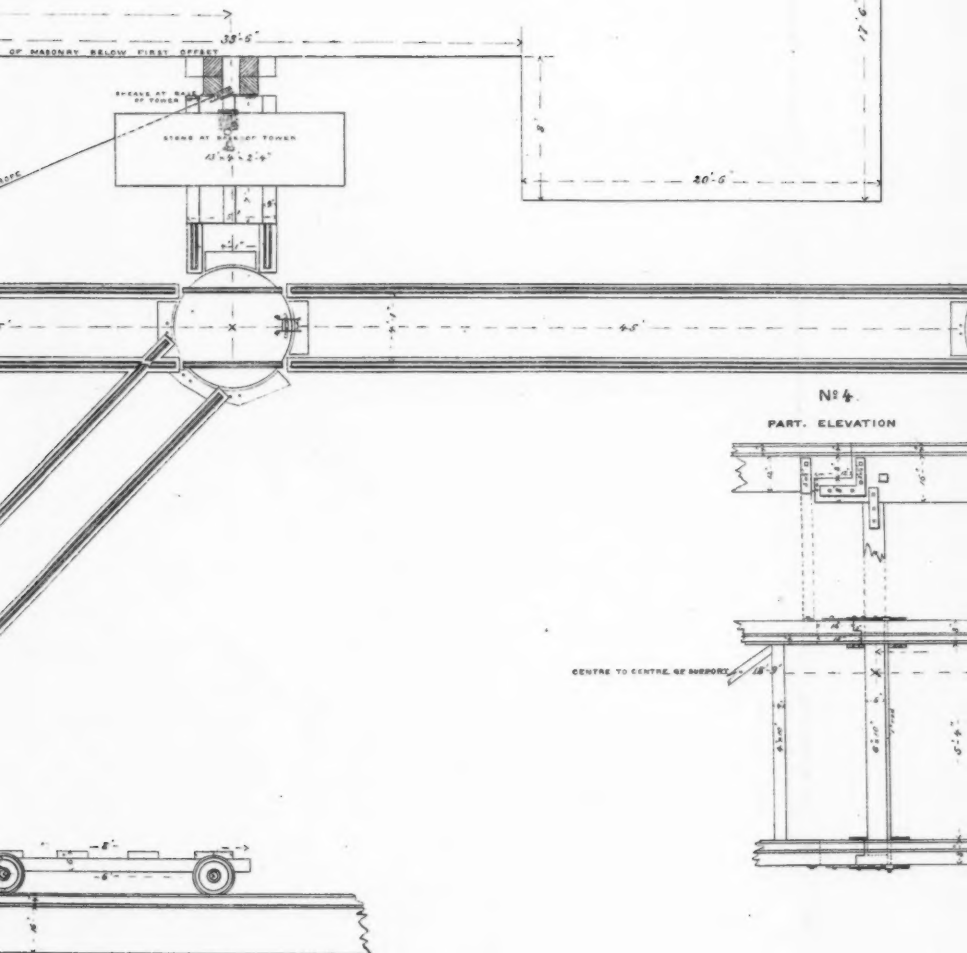


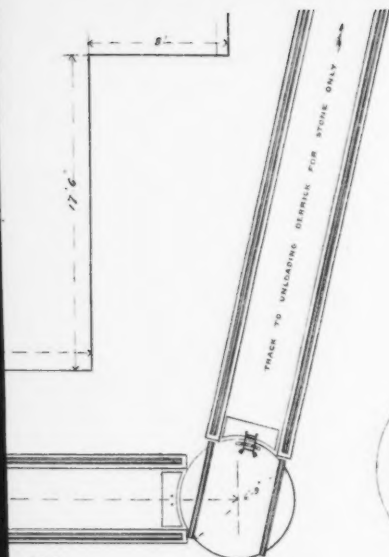
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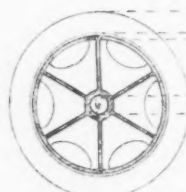
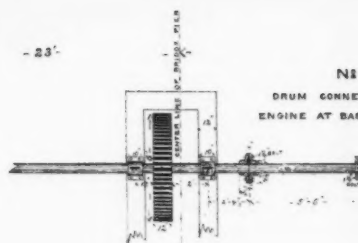
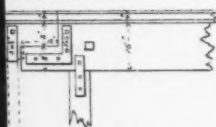
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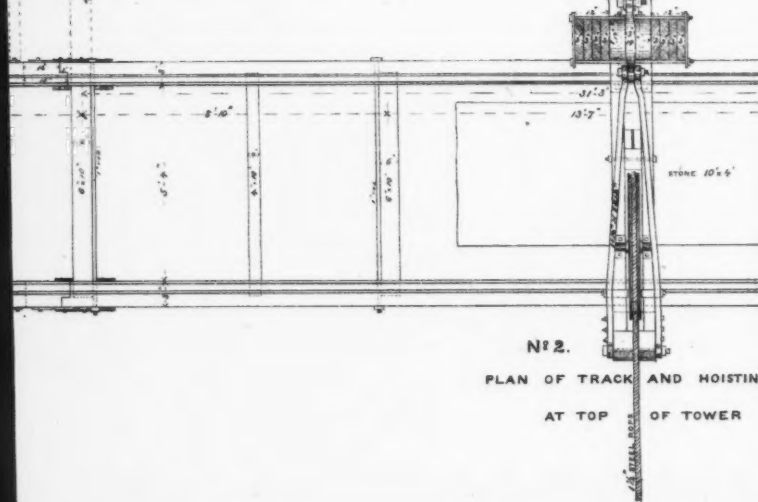
Nº 4.

ELEVATION



Nº 8.

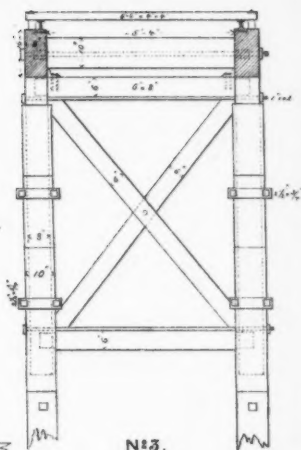
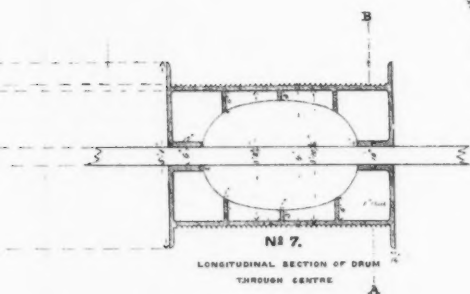
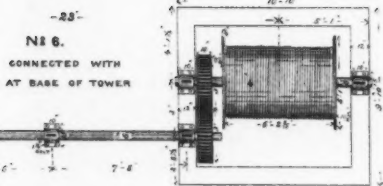
CROSS SECTION OF DRUM
AT A.B.



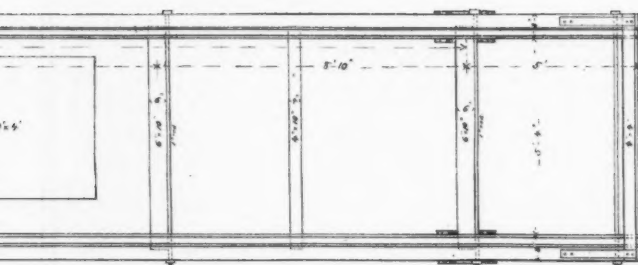
Nº 2.

PLAN OF TRACK AND HOISTING

AT TOP OF TOWER



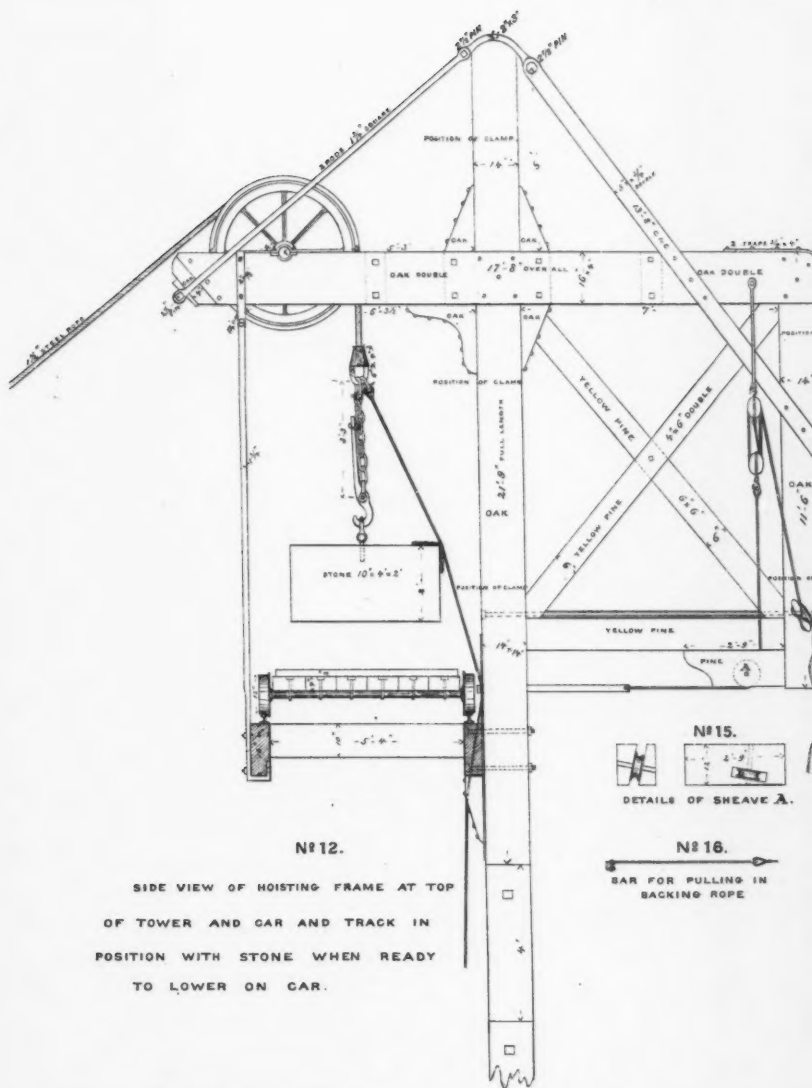
END ELEVATION OF TRACK AND
SUPPORTING FRAME AT TOP OF
TOWER



DISTING FRAME

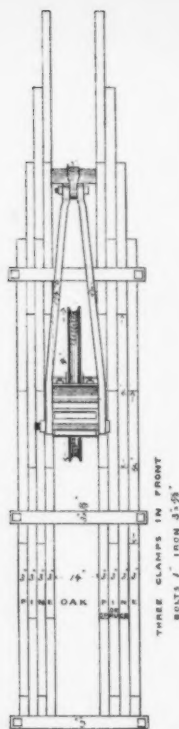
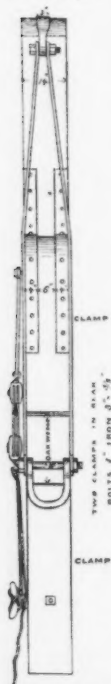
OVER





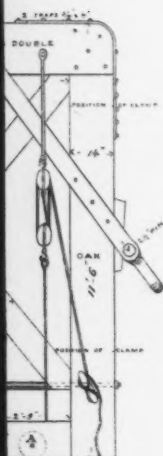
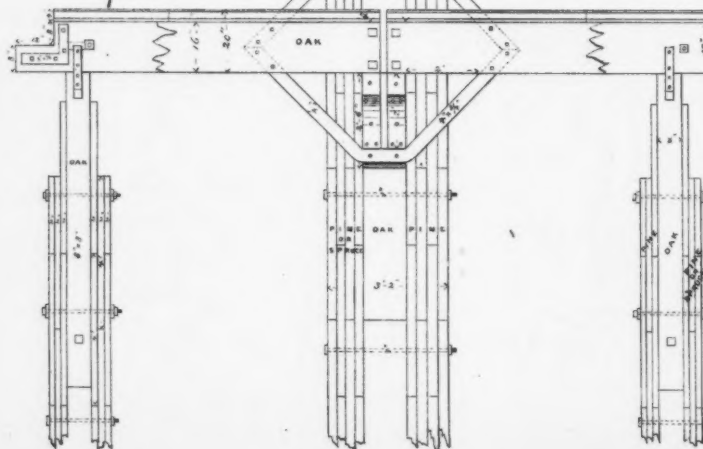
N^o14.

REAR VIEW



N^o13.

ELEVATION OF PART OF
SUPPORTING FRAME AT TOP



5.



HEAVE A.

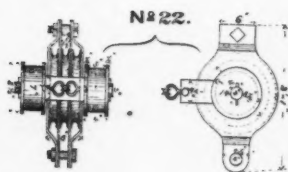
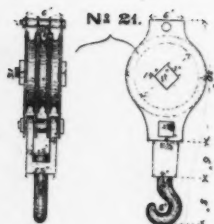
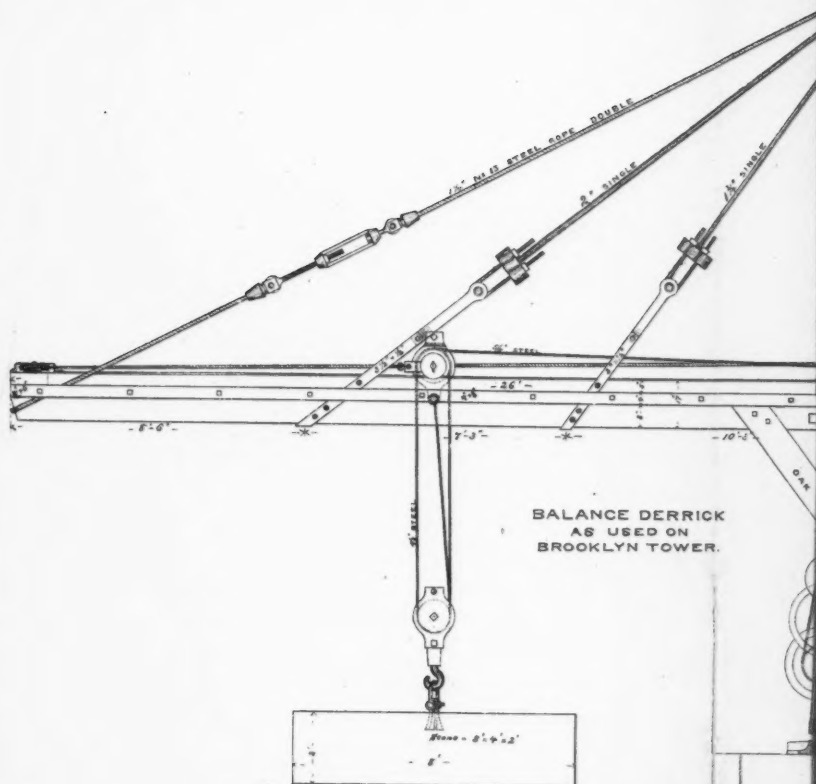
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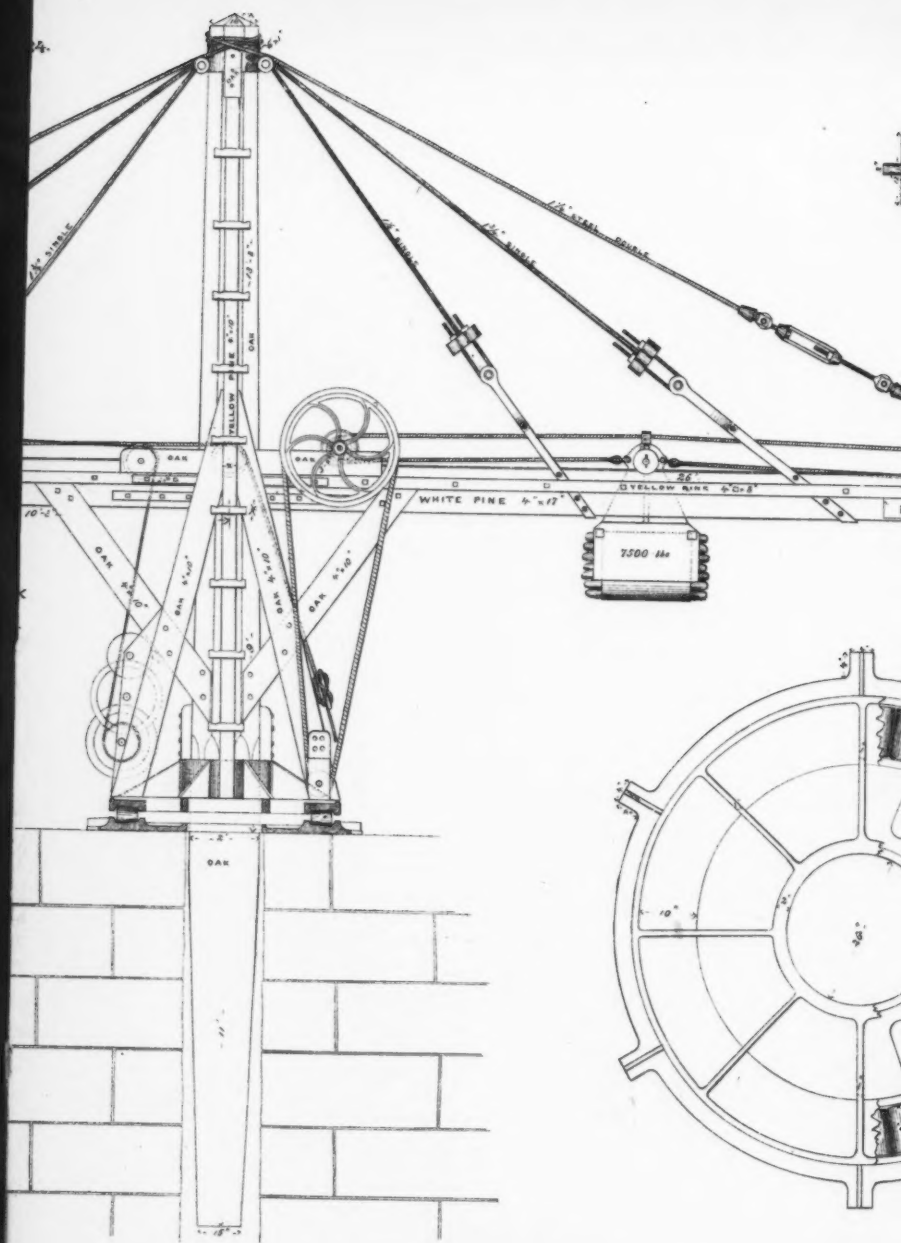
Fig. 13.

SECTION OF TRACK AND
SECTION OF TOWER

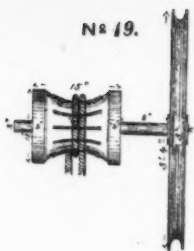


N^o 24.

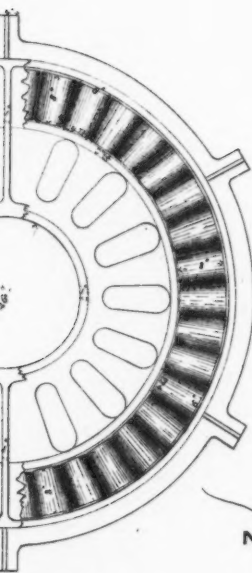
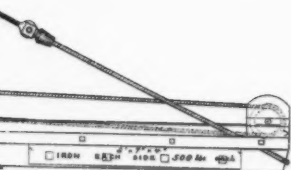
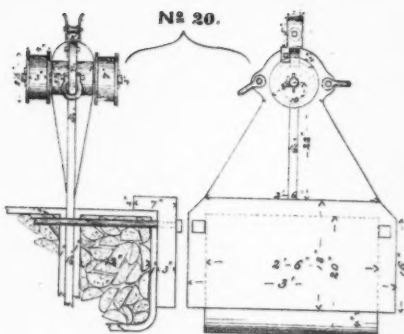




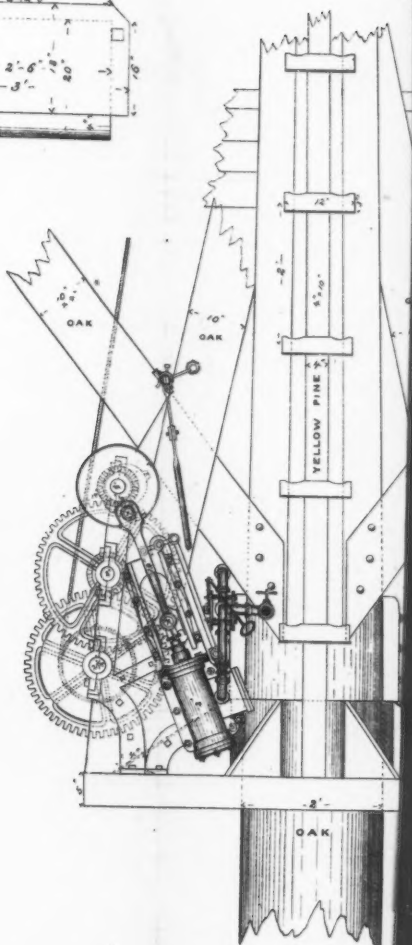
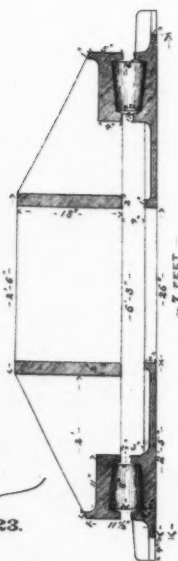
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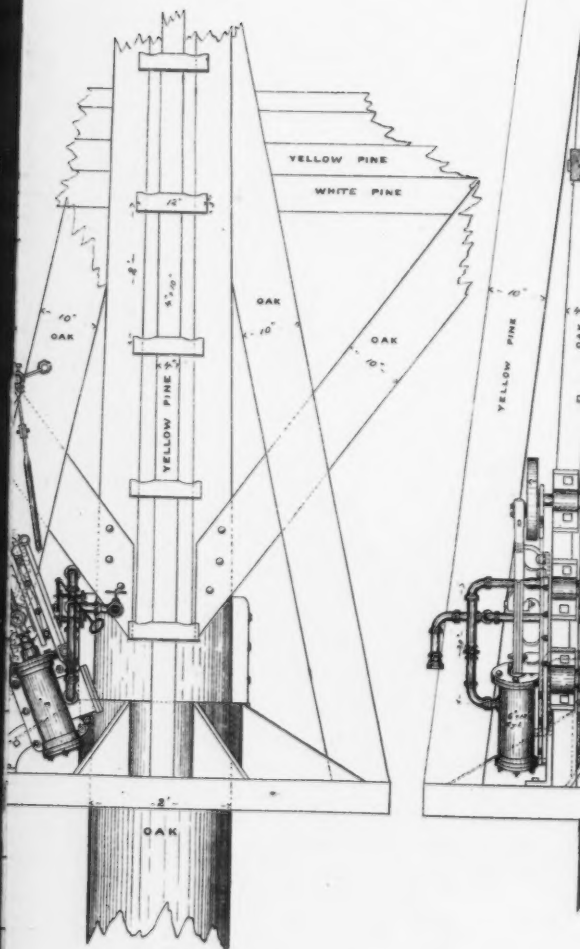
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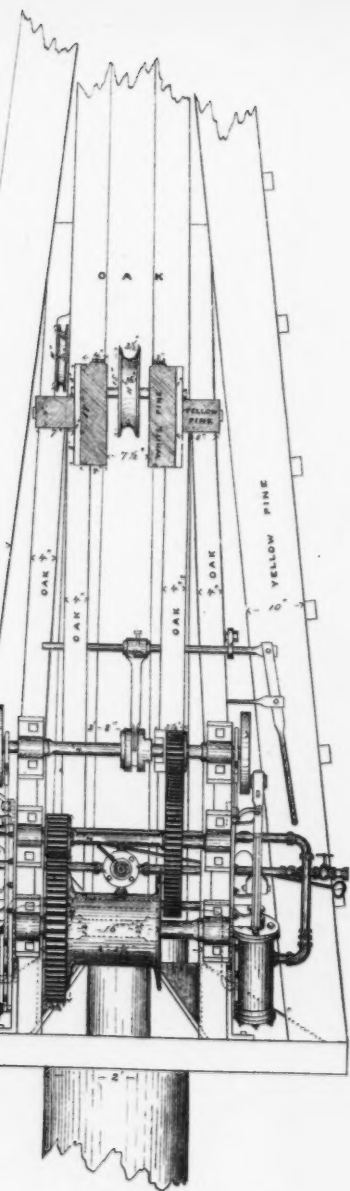
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N^o 47.

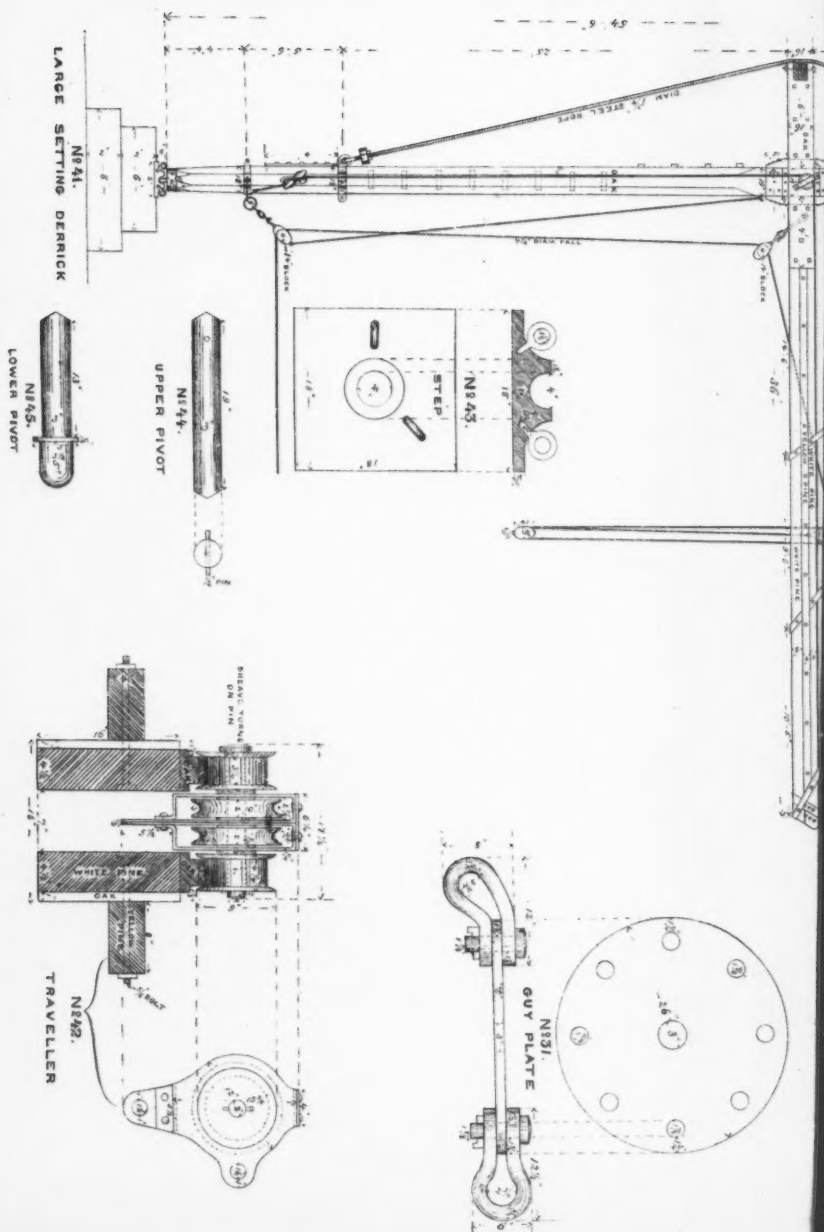


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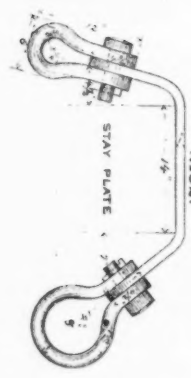
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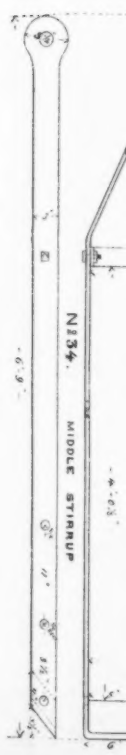




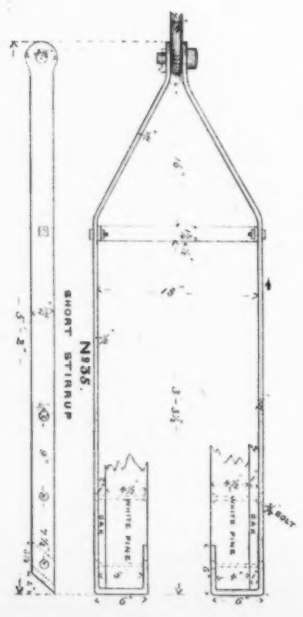
STAY PLATE BEFORE BENDING



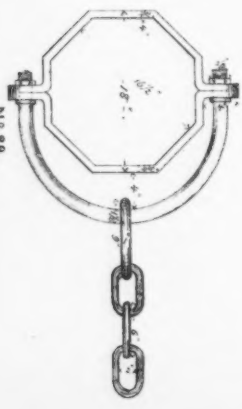
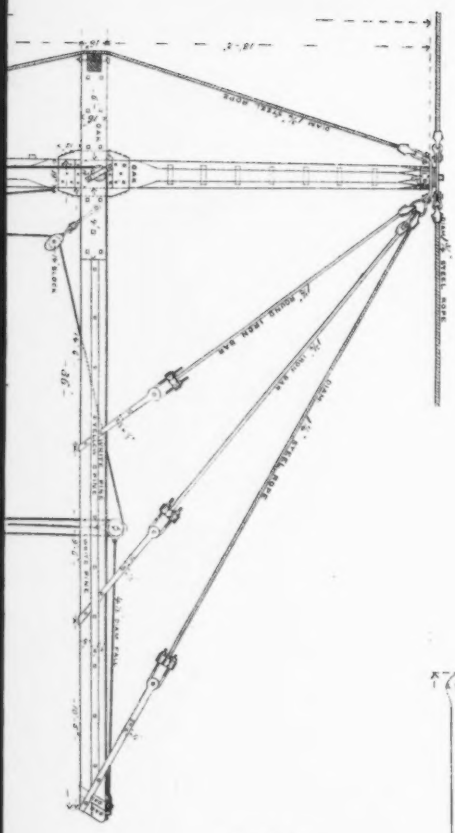
N832.



N834. MIDDLE STIRRUP



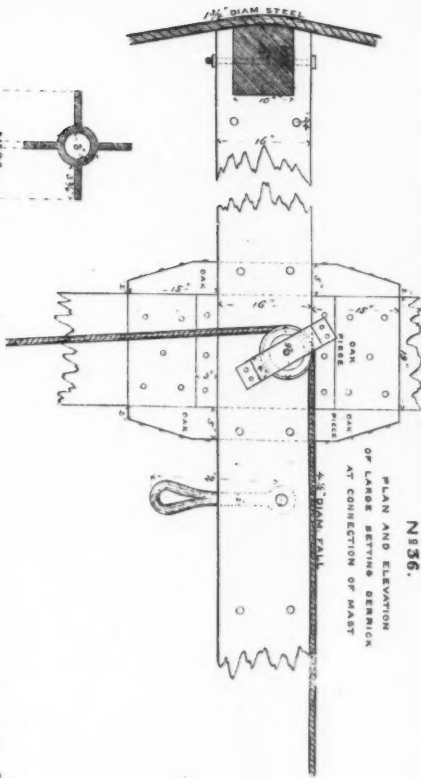
N835. SHORT STIRRUP



N829. TRAVELLING RING

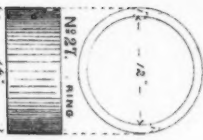
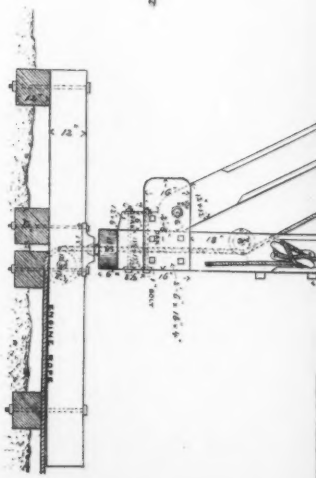
N336.

PLAN AND ELEVATION
OF LARGE SETTING DERRICK
AT CONNECTION OF MAST

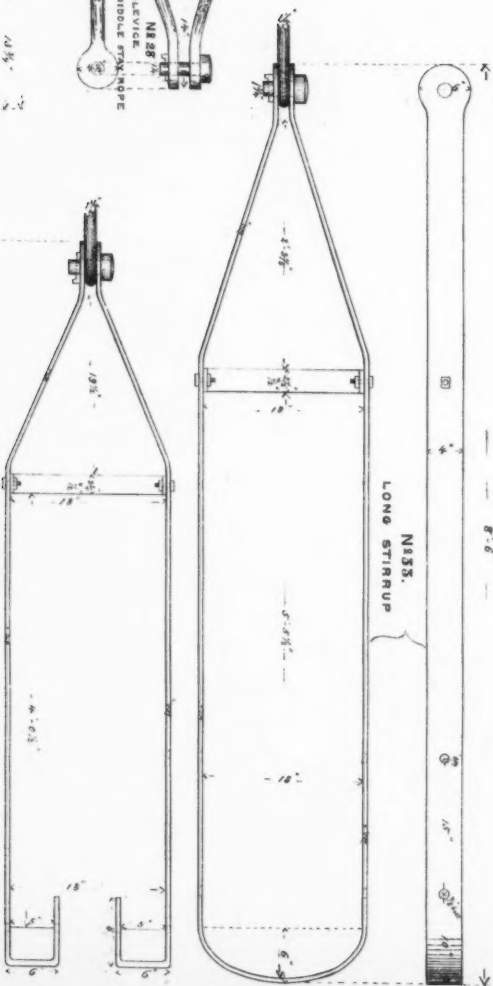


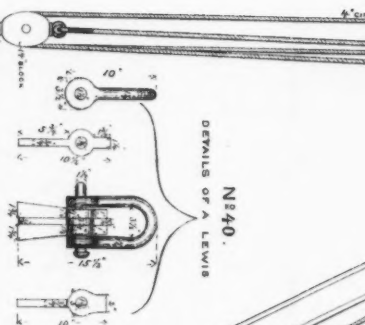
UNLOADING DERRICK

N337.

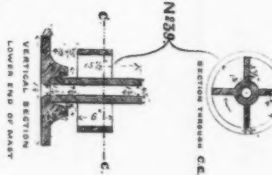


N333.
LONG STIRRUP



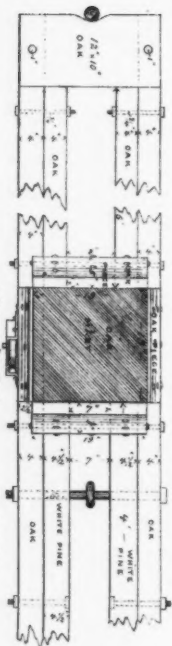
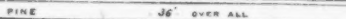


N^o 40.



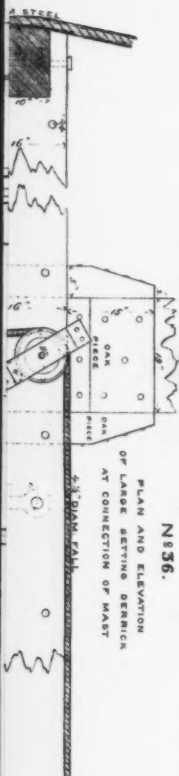
N839

SECTION THROUGH C.E.



N 836

PLAN AND ELEVATION OF LARGE SETTING DERRICK AT CONNECTION OF MAST



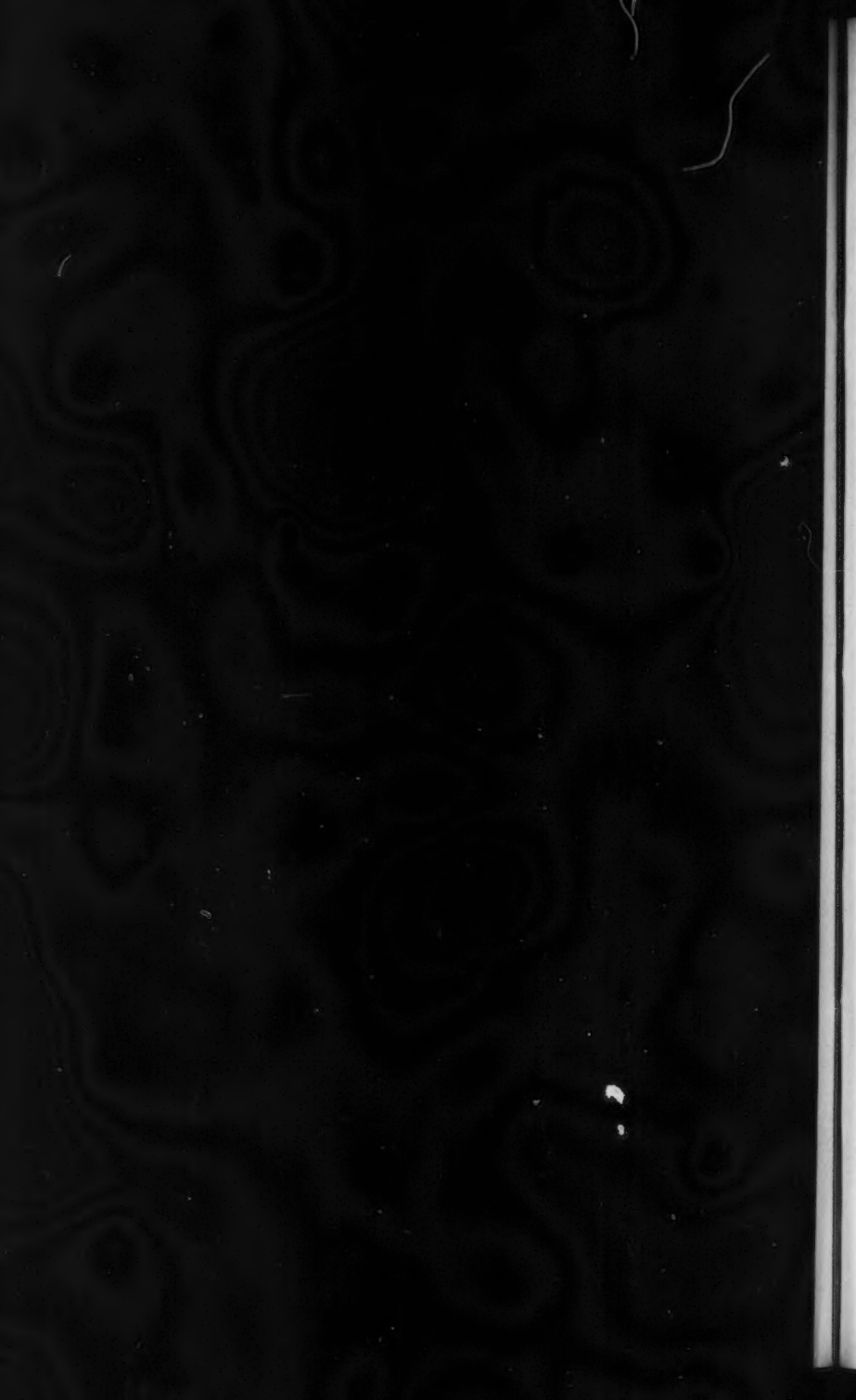


PLATE IV. Fig. 12.—Side view of hoisting frame at top of tower; car and track in position, with stone when ready to lower on car.

Fig. 13.—Elevation of part of track and supporting frame, at top of tower.

Fig. 14.—Rear view of frame.

“ 15.—Details of sheave.

“ 16.—Bar for pulling backing rope in.

PLATE V. Figs. 17, 18.—Engine on balance derrick.

Fig. 19.—Drum for balance weight rope.

“ 20.—Balance weight.

“ 21.—Iron block for hoisting.

“ 22.—Traveller.

“ 23.—Bedplates of engine on balance derrick.

“ 24.—Balance derrick as used on Brooklyn tower.

PLATE VI. Fig. 26.—Spider.

“ 27.—Ring at top and bottom of mast.

“ 28.—Clevis for middle stay-rope.

“ 29.—Travelling ring.

“ 30.—Back-stay ring.

“ 31.—Guy plate.

“ 32.—Stay plate.

Figs. 33, 34, 35.—Long, middle and short stirrups.

Fig. 36.—Plan and elevation of large setting derrick, at connection of mast.

Fig. 37.—Unloading derrick.

Figs. 38, 39.—Sheaves at upper and lower end of mast.

Fig. 40.—Details of lewis.

“ 41.—Large setting derrick.

“ 42.—Traveller on boom.

“ 43.—Footstep.

Figs. 44, 45.—Upper and lower pivots.

THE RATE OF SET OF METALS SUBJECTED TO STRAIN

FOR CONSIDERABLE PERIODS OF TIME.

A Paper by Prof. ROBERT H. THURSTON, Member of the Society.

READ DECEMBER 6TH, 1876.

SECTION I.*—ON THE OBSERVED DECREASE OF RESISTANCE AT A FIXED DISTORTION.—The writer has, in a preceding paper † shown, by reference to experimental researches, in which he had then engaged, that some classes of metals, as ordinary iron and steel, when subjected to strain and distortion by a force exceeding the resistance of the material within the elastic limit, take a set and are stiffened by that act, and exhibit an exaltation of the elastic limit. It was also shown that other classes, like tin, and similarly viscous and ductile materials, exhibit flow and a depression of their limits of elasticity when similarly treated.‡ It was further shown, that the former class when subjected to loads, even approaching their ultimate strength, took a certain set and remained apparently indefinitely without further distortion; while the second class, under very moderate loads, frequently exhibit a gradual yielding, a progressive distortion, until fracture took place, sometimes under stresses which were but a fraction of those which were found required to break such metals quickly, and when time was not allowed for flow to occur. It was noted that increase of rapidity of distortion and fracture produced increase of resistance in the latter, or "tin-class," and decrease of resisting power in the first, or "iron-class," and *vice-versa*.

The writer has since instituted experiments upon metals of both classes to determine how rapidly set, in each class, took place; the earlier experiment just referred to, having confirmed a suspicion long existing among engineers and experimentalists, that the phenomenon was a molecular change, as well as of the mass, and that time was required for its complete development. Prof. Norton has also shown by experiment that this set is partially temporary, the bar relieving itself of distortion in some degree, on removal of the load. Both that experi-

* Prepared July, 1876.

† CXXIII. Note on the Resistance of Materials as affected by Flow, and by Rapidity of Distortion. Vol. V. page 199; also Van Nostrand's Engineering Magazine, September, 1876, and Engineering (London), December 29, 1876.

‡ Mr. E. H. Hewins has informed the writer since the publication of that paper, that he has detected simultaneous "flow," and "exaltation of the elastic limit" in iron.

menter and the writer had detected some peculiar variations of form during this recovery, and the experiments of the latter, as detailed in the preceding paper, exhibited at times a gradual recovery of straightening power in a confined and flexed bar. The following will be found interesting, and perhaps, important, as showing how these molecular changes progress.

Bars were prepared of square section, 1 inch in breadth and depth, and 22 inches in length, between bearings. They were flexed in a machine* for testing the resistance of materials to transverse stress, as described in the preceding paper and the load and deflection carefully measured. As the bars were retained at a constant deflection, their effort to resume their original form gradually decreased, and the amount of this effort was, from time to time noted. When this effort or resistance had become considerably decreased, the bar was released, and the set measured. This operation was repeated with each, until the law of decrease of elastic resistance was detected. Curves were constructed, illustrating graphically this law, and exhibiting it more satisfactorily and more plainly than the tabular record.

The following is the record for the bars of iron, of tin, and of two alloys. The iron bar No. 648, was subjected to a load 1 003 pounds, somewhat less than one-half its maximum, and its deflection was found to be 0.0995 inch. Removing the load, the set was 0.0049 inch. Restoring the load (1 000 pounds, + 3 pounds due to the weight of the bar), the deflection was 0.1001 inch, and the bar was held at this deflection and the decrease of resistance observed. In 25 minutes, it had become 999 pounds; in 1 hour 40 minutes, 991 pounds; in 4 hours 35 minutes, 987 pounds, and in 5 hours 20 minutes, 987 pounds. The set was then found to be 0.007 inch under the weight of the bar itself.

Restoring the last observed load, the deflection was 0.0991 inch, and the original load of 1 003 pounds increased it to 0.1003 inch.

A second trial of the same bar under a load of 1 603 pounds gave a deflection of 0.2548 inches, and a set, on removal, of 0.1091 inch. Restoring the load, the deflection became 0.287 inch, and the resistance to flexion decreased in 6 hours 3 minutes, from 1 603 to 1 457 pounds, at which latter time the set was found to be 0.1451 inch. Restoring the load of 1 457 pounds, the deflection was 0.2863 inch, and the original load, 1 603 pounds, being brought upon it, its deflection increased to 0.3016 inch, an increase nearly 20 per cent. above the original deflection.

* Built for the Mechanical Laboratory of the Stevens Institute of Technology.

In the first trial the loss of stiffness, as measured by the decrease of effort to straighten itself, and which is here taken to measure the *rate of set*, is seen to have been nearly proportional to the time at first, becoming constant after $4\frac{1}{2}$ hours. On the second trial, after a considerable set, produced by a heavy load, the set became constant after about one hour, and so remained to the end of the trial.

No. 655 was a bar of Queensland tin, received* from the Commissioner of that country at the Centennial Exhibition, and which was found to be remarkably pure. A load of 100 pounds gave a deflection of 0.2109 inch, and produced a set of 0.1753 inch. The same load restored, deflected the bar 0.2415 inch, which deflection being retained, the effort to regain the original shape decreased in one minute from 100 to 70 pounds, in 3 minutes to 62, and in 8 minutes to 56 pounds. The original load of 100 pounds then brought the deflection to 0.3033 inch, nearly 50 per cent. more than at first.

A bar, No. 599, of copper-zinc alloy similarly tested, deflected 0.5209 inch under 1 233 pounds, and took a set of 0.2736 inch after being held at that deflection 15 minutes, the effort falling, meantime to 1 137 pounds. Restoring the load of 1 137 pounds, the deflection became 0.5131 inch, and the original load of 1 233 pounds brought it to 0.5456 inch. The bar was now held at this deflection and the set gradually took place, the effort falling in 15 minutes to 1 133 pounds—4 per cent. more than at the first observation—in 22 minutes to 1 093, in 46 minutes to 1 063, in 63 minutes to 1 043, in $91\frac{1}{2}$ minutes to 1 003, and in 118 minutes to 911 pounds; at which last strain the bar broke 3 minutes later, the deflection remaining unchanged up to the instant of fracture. This remarkable case has already been referred to in an earlier paper,† when treating of the effect of time in producing variation of resistance and of the elastic limit.

Nos. 561, copper-tin, and 612, copper-zinc, were compositions which behaved quite similarly to the iron bar at its first trial, the set apparently becoming nearly complete in the first after 1 hour, and in the second after 3 or 4 hours.

In all of these metals, the set and the loss of effort to resume the original form, were phenomena requiring time for their progress, and in all, except in the case of No. 599—which was loaded heavily—the change gradually became less and less rapid, tending constantly toward a maximum.

* By the Mechanical Laboratory of the Stevens Institute of Technology.

† Vol. V, page 205.

So far as the observation of the writer has yet extended, the latter is always the case under light loads. As heavier loads are added, and the maximum resistance of the material is approached, the change continues to progress longer, and, as in the case of the brass above described, it may progress so far as to produce rupture, when the load becomes heavy, if the metal does not belong to the "iron-class." The brass broke under a stress 25 per cent. less than it had actually sustained previously.

There is no evidence that iron or steel ever exhibits this treacherous and exceedingly dangerous behavior; but, on the contrary, it seems always to carry a load, once borne, however near the maximum it may be. This difference is here, quite as marked as in the experiments previously reported, upon the elevation and the depression of the elastic limit by strain; and no one can fail to note the value in construction of this quality of that metal which is the chief reliance of the engineer in nearly every branch of his art. These principles will find numberless applications in the practice of every member of the profession.

The records are herewith presented, and the curves representing them, shown in Plate VII.

RECORDS OF EXPERIMENTS ON RATE OF SET OR DECREASE OF RESISTANCE
AND INCREASE OF SET OF METALS WITH TIME.
Bars 1 inch square, 22 inches between supports.

TIME.	LOAD.	LOSS OF LOAD.	DEFLEC- TION.	SET.	TIME.	LOAD.	LOSS OF LOAD.	DEFLEC- TION.	SET.
Minutes	Pounds.	Pounds.	Inches.	Inches.	Minutes	Pounds.	Pounds.	Inches.	Inches.
No. 648. WROUGHT IRON.					3	0.1091
First Trial.					1 603	0.287
....	1 003	0.0995	1	1 521	82	0.287
....	3	0.0049	2	1 493	110	0.287
....	1 003	0.1001	3	1 483	129	0.287
25	999	4	0.1001	23	1 463	140	0.287
100	991	12	0.1001	53	1 461	142	0.287
275	987	16	0.1001	133	1 459	144	0.287
320	987	16	0.1001	193	1 457	146	0.287
320	3	0.007	363	1 457	146	0.287
322	987	0.991	363	3	0.1481
322	1 003	0.1003	1 457	0.2833
....	2 720	2.64	1 603	0.3016
Second Trial.					2 720	2.64
....	1 003	2.2548

RECORDS.—(Continued.)

TIME.	LOAD.	LOSS OF LOAD.	DEFLEC- TION.	SET.	TIME.	LOAD.	LOSS OF LOAD.	DEFLEC- TION.	SET.
Minutes	Pounds.	Pounds.	Inches.	Inches.	Minutes	Pounds.	Pounds.	Inches.	Inches.
No. 561. 27.5 PARTS COPPER, 72.5 PARTS TIN.					96.5	993	240	0.5456
....	160	0.0696	118	911	322
....	5	0.0145	121	911	326	Broke.
....	160	0.072	No. 612. 47.5 PARTS COPPER, 52.5 PARTS ZINC.				
1	154	6	0.072	800	0.3332
3	150	10	0.072	3	0.1478
2 640	104	56	0.072	800	0.3366
4 140	100	60	0.072	5	790	10	0.3366
....	5	0.04	25	778	22	0.3366
....	160	0.0763	120	766	34	0.3366
....	160	0.097	480	756	44	0.3366
....	320	0.22	Broke.	1 320	751	49	0.3366
No. 599. 10 PARTS COPPER, 90 PARTS ZINC.					3	0.1688
....	1 233	0.5209	751	0.3364
15	1 137	0.5209	800	0.349
....	3	0.2736	1 100	Broke.
....	1 137	0.5131	No. 655. QUEENSLAND TIN.				
....	1 233	0.5456	100	0.2109
15	1 133	100	0.5456	3	0.1753
28	1 093	140	0.5456	100	0.2415
40	1 070	163	0.5456	1	70	30	0.2415
46	1 063	170	0.5456	3	62	38	0.2415
63	1 043	190	0.5456	8	56	44	0.2415
77.5	1 023	210	0.5456	100	0.3033
91.5	1 003	230	0.5456	150	Bent rapidly.	

SECTION II*—THE OBSERVED INCREASE OF DEFLECTION UNDER STATIC LOAD.—In the preceding section, the writer presented results of an investigation made† to determine the time required to produce "set" in metals belonging to the two typical classes, which exhibit, the one an exaltation and the other a depression of the elastic limit under strain.

The experiments there described, were made by means of a testing machine in which the test piece could be securely held at a given degree of distortion, and its effort to recover its form measured at intervals,

* Prepared November, 1876.

† In the Mechanical Laboratory of the Stevens Institute of Technology.

until the progressive loss of effort could no longer be detected, and until it was thus indicated that set had become complete.

The deductions were :

That in metals of all classes, under light loads, this decrease of effort and rate of set become less and less noticeable until, after some time, no further change can be observed, and the set is permanent :

That in metals of the "tin-class," or those which had been found to exhibit a depression of the elastic limit with strain, a heavy load. *i. e.*, a load considerably exceeding the proof-strain, the loss of effort continued until, before the set had become complete, the test piece yielded entirely :

And that in the metals of the "iron-class," or those exhibiting an elevation of elastic limit by strain, the set became a maximum and permanent and the test-piece remained unbroken, no matter how near the maximum load the strain may have been.

The experiments here described were conducted with the same object as those above referred to. In these experiments, however, the load, instead of the distortion, was made constant, and deflection was allowed to progress, its rate being observed, until the test-piece either broke under the load or rapidly yielded, or until a permanent set was produced. It will be seen that the results of these experiments are in striking accordance with those conducted in the manner previously described ; they exhibit the fact of a gradually changing rate of set for the several cases of light or heavy loads, and illustrate the striking and important distinctions between the two classes of metals even more plainly than the preceding. The accompanying record and the strain-diagrams,¹ (Plate VIII), which are its graphical representation, will assist the reader in comprehending the method of research and its results. All test-pieces were of one-inch square section, and loaded at the middle. The bearings were 22 inches apart.

No. 651 was of wrought iron from the same bar with No. 648, already described.* This specimen subsequently gave way under a load of 2 587 pounds. Its rate of set was determined at about 60 per cent. of its ultimate resistance, or at 1 600 pounds. Its deflection, starting at 0.489 inch, increased in the first minute 0.1047, in the second minute 0.026, in the third minute 0.0125, in the fourth minute 0.0088, in the fifth minute 0.0063, and in the sixth minute 0.0031 inch ; the total deflections being 0.5937, 0.6197, 0.6322, 0.641, 0.6473, and 0.6504 inch. In the succeeding 10 minutes the deflection only increased 0.0094 inch,

* Vol. V, Page 208.

or to 0.6598 inch, and remained at that point without increasing so much as 0.0001 inch, although the load was allowed to remain 344 minutes untouched. The bar had evidently taken a permanent set, and it seems to the writer probable, that it would have remained at that deflection indefinitely, and have been perfectly free from liability to fracture for any length of time.

This bar finally yielded completely, under a load of 2 589 pounds, deflecting 4.67 inches.

No. 479 was a copper bar containing $3\frac{1}{4}$ per cent. of tin. Its behavior may be taken as typical of that of the whole "tin-class" of metals, as the preceding illustrates the behavior of the "iron-class" under heavy loads. It was subjected to two trials, the one under a load of 700 and the other of 1 000 pounds, and broke under the latter load, after having sustained it $1\frac{1}{2}$ hours. The behavior of this bar will be considered especially interesting, if its record and strain-diagram are compared with those of No. 599, previously given, which latter specimen broke after 121 minutes when held at a constant deflection of 0.5456 inch; its resistance gradually falling from an initial amount of 1 233 pounds, to 911 pounds at the instant before breaking.

This bar, No. 479, was loaded with 700 pounds "dead weight," and at once deflected 0.441 inch. The deflection increased 0.118 inch in the first 5 minutes, 0.024 in the second 5 minutes, 0.018 in the second 10 minutes, 0.17 in the fourth, 0.012 in the fifth, and 0.008 inch in the sixth 10 minute-period, the total set increasing from 0.441 to 0.65 inch. The record and the strain-diagram, (Plate VIII) show that, at the termination of this trial, the deflection was regularly increasing. The load was then removed and the set was found to be 0.524 inch, the bar springing back 0.126 inch on removal of the weight.

The bar was again loaded with 1 000 pounds. The first deflection which could be caught and measured, was 3.118 inches and the increase at first followed the parabolic law noted in the preceding cases, but quickly became accelerated; this sudden change of law is best seen on the strain-diagram. The new rate of increase continued until fracture actually occurred, at the end of $1\frac{1}{2}$ hours, and at a deflection of 4.506 inches.

This bar was of very different composition from No. 599; it is a member of the "tin-class," however, and it is seen, by examining their records and strain-diagrams, that these specimens, tested under radically different conditions, both illustrate the peculiar characteristics of the class, by similarly exhibiting its treacherous nature.

No. 504 was a bar of tin containing about 0.6 per cent. of copper—the opposite end of the scale—and exhibited precisely similar behavior, taking a set of 0.323 inch under 110 pounds and steadily giving way and deflecting uninterruptedly until the trial ended at the end of 1 270 minutes, over 21 hours. This bar, subsequently, was, by a maximum stress of 130 pounds, rapidly broken down to a deflection of 8.11 inches.

No. 501 presents the finest illustration yet entered in the record book of the Mechanical Laboratory of the Stevens Institute of Technology. The test extended over nearly $2\frac{1}{2}$ days under observation, and then left for the night, was found next morning broken. The time of fracture is therefore unknown, as is the ultimate deflection. The record is, however, sufficient to determine the law, and the strain-diagram (Plate VIII) is seen to be similar to that of the second test of No. 479, exhibiting the same tendency to the parabolic shape and the same change of law and reversal of curvature preceding final rupture, and illustrates even more strikingly the fact that this class of metals is not safe against final rupture, even though the load may have been borne a considerable time, and have apparently been shown, *by actual test*, to be capable of sustaining it. A strain-diagram of each of the latter two bars is exhibited on a reduced scale, to present to the eye, more strikingly, this important characteristic. (Plate VIII.)

A comparison of the records (next page) and the strain-diagrams (Plate VIII), with those of Section I, in illustration of the behavior of the two classes of metals under constant deflection, is most instructive. The light thus thrown upon the phenomena of distortion and fracture may be of great service to all who are engaged in construction. It will be necessary to make many experiments to determine under what fraction of their ultimate resistance to rapidly applied and removed loads, the members of the "tin-class"—the viscous metals—will be safe under static permanent loads. Their behavior under shocks of various intensities remains also to be determined. The most probable and most satisfactory conclusion which seems likely to be finally reached is, perhaps, that the "iron-class" of metals are capable of carrying indefinitely any load which they have once borne, and that, in some manner—by the relief of internal strain as suggested by the writer* or by some other process—their rest under a load renders them, as time goes on, more and more safe under that load.

* Wire makers have learned that newly made wire is considerably weaker than similar wire which has been so long made as to afford time for relief, by flow, of the internal straining introduced by the process of drawing.

RECORD OF EXPERIMENTS WITH "DEAD LOADS" TO DETERMINE THE INCREASE
OF DEFLECTION WITH TIME, OR RATE OF SET.

Bars, 1 inch square, 22 inches between supports.

Load applied at the middle.

TIME.	DEFLEC- TION.	INCREASE. Inches.		TIME.	DEFLEC- TION.	INCREASE. Inches.	
Minutes.	Inches.	Difference.	Total.	Minutes.	Inches.	Difference.	TOTAL.
No. 651. WROUGHT IRON. Load, 1 600 pounds.				40	0.63	0.012	0.189
0	0.489	50	0.642	0.012	0.201
1	0.5937	0.1047	0.1047	60	0.65	0.008	0.209
2	0.6197	0.026	0.1307	Set.	0.524
3	0.6322	0.0125	0.1432	Second Trial.—Load, 1 000 pounds.			
4	0.641	0.0088	0.152	0	3.118
5	0.6473	0.0063	0.1583	5	3.54	0.422	0.422
6	0.6504	0.0031	0.1614	15	3.66	0.12	0.542
16	0.6598	0.0094	0.1708	45	4.102	0.442	0.984
344	0.6598	0.0000	0.1708	75	7.634	3.522	4.506
Maximum load, 2589 pounds; maximum deflection, 4.67 inches.				Bar broke under 1000 pounds.			
No. 504. 0.557 PARTS COPPER, 99.443 PARTS TIN. Load, 110 pounds.				No. 501. 9.7 PARTS COPPER, 90.3 PARTS TIN. Load, 160 pounds.			
0	0.323	0	1.294
5	0.406	0.083	0.083	10	1.319	0.025	0.025
845	1.945	1.539	1.622	70	1.463	0.144	0.169
865	2.005	0.059	1.681	130	1.53	0.067	0.236
895	2.138	0.134	1.815	310	1.691	0.161	0.397
1 025	2.248	0.11	1.925	400	1.766	0.075	0.472
1 110	2.378	0.13	2.055	460	1.811	0.045	0.517
1 270	2.626	0.248	2.303	1 360	2.534	0.723	1.24
Maximum load, 130 pounds; maximum deflection, 8.11 inches.				1 475	2.697	0.163	1.403
No. 479. 96.27 PARTS COPPER, 3.73 PARTS TIN. Load, 700 pounds.				1 565	2.782	0.085	1.488
0	0.441	1 730	2.938	0.156	1.644
5	0.559	0.118	0.118	1 880	3.136	0.198	1.842
10	0.583	0.024	0.142	2 780	3.798	0.662	2.504
20	0.601	0.018	0.16	2 940	4.274	0.476	2.98
30	0.618	0.017	0.177	3 000	4.349	0.075	3.055
				3 295	5.097	0.748	3.803
				Bar left under strain at night and found broken in the morning.			

The law of deflection and of rate of set, as illustrated graphically by the strain-diagrams given in this and in the preceding paper, is expressed for the lighter loads by equation of the form

$$Y=AT-BT^2$$

in which Y is the deflection or the set, both quantities varying together in this case, and T is the time; A and B being constant co-efficients to be determined for special cases.

For heavy loads, after the first sudden deflection and set, the equation is seen to be

$$Y=AT$$

in which for iron, $A=\frac{1}{Y}$ and for the tin-class A is a constant multiplier up to a limit x , Nos. 501, 479, at which it varies as some new function of the time.

The values of constants for the various metals remain to be determined. The question whether this change in the value of the Modulus of Rupture, as exhibited in the preceding section, and of the value of the quantity representing in the usual formulas the amount of deflection is due to a change in the Modulus of Elasticity, to simple flow, or to a variation of cohesive force, remains to be considered.

SECTION III.*—ELEVATION OF ELASTIC LIMIT IN GUN BRONZE.—The writer would refer to the recent criticisms of Prof. Kick, of Prague, on "Autographically produced Strain-Diagrams, and the Elevation of the elastic Limit by Strain." In a late issue of *Dingler's Polytechnisches Journal*,† the Professor, in rejoinder to my reply‡ to his criticism of my paper on the subject of the strength of materials and the elevation of the elastic limit by strain, asserts:

1°. That I use his formula, for determining the errors of apparatus due to velocity of motion, incorrectly.

2°. That I claim to be able to deduce the amount of work done in deformation of the test-piece from automatically produced diagrams in which the abscissas are proportional to the angular motion of the handle.

3°. That the error introduced by a blow will be greater as velocities are greater.

4°. That experimental proof of the elevation of the elastic limit in gun-bronze by strain was presented to Prof. Kick and others in August, 1873, by Gen. Uchatius, and that his discovery antedates that announced by me to this Society in November ½ of that year.

* Prepared October, 1876.

† Dingler's Polytechnisches Journal, Band 220; Heft 2. ‡ CXVII, Vol. V, page 9.

§ LXI. A Note on the Resistance of Materials, Vol. II, page 239.

I have been prevented by illness from noticing that statement before. I would now say:

1°. That I used his formula purposely as its author applied it, in criticising my paper, in order to make more striking the refutation. My reply is just as complete as if I had applied it to the more intricate case.

2°. That the abscissas of the strain-diagrams produced automatically are not proportional to the "motion of the handle," but to the distortion of the test-piece, and that this singular misapprehension of the subject of the criticism may be the result of that which prompted the original criticism.

3°. That I have distinctly disclaimed all intention of ascribing to the Autographic Recording Testing Machine the power of giving *quantitative* results when affected by shocks, and that my paper stands perfectly good, notwithstanding this fact, which was there implicitly stated, and would have been explicitly stated had it not seemed so perfectly obvious.

4°. That I shall endeavor, health and time permitting, to present proof that the experiments of Gen. Uchatius, in 1873 and before the date of my paper, do not prove an elevation of the elastic limit by strain; and, finally,

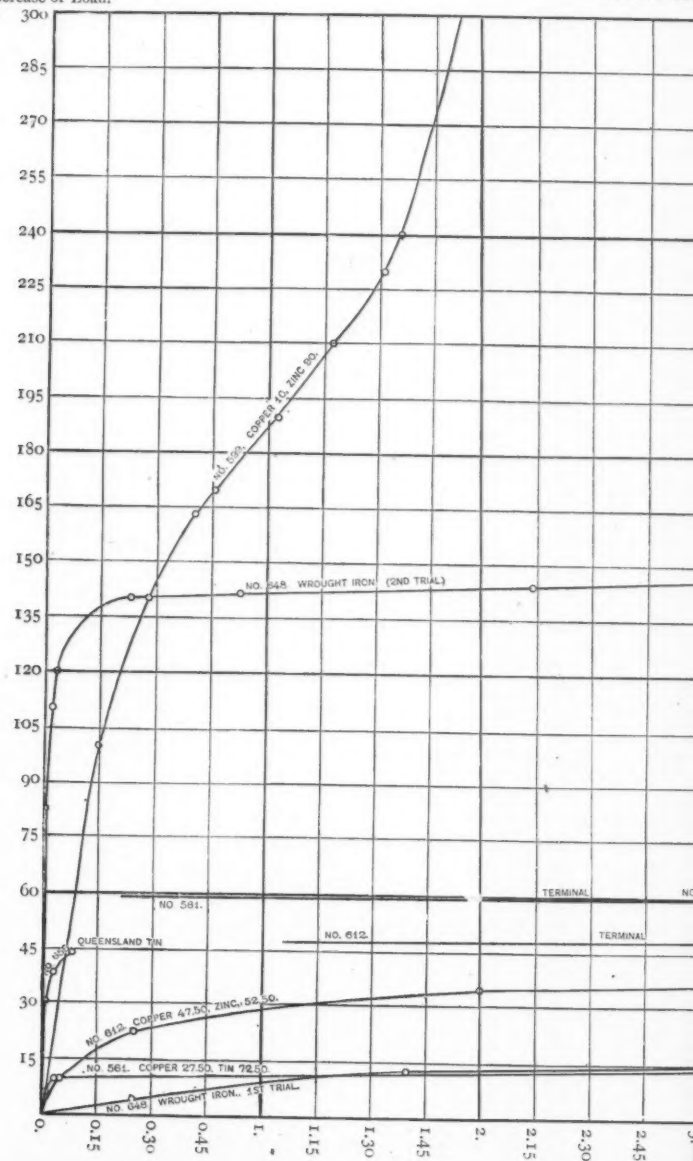
5°. That *gun-bronze does not possess this property.*

The phenomenon there shown was due, I think, simply to that condensation of metal by pressure, such as occurs in Whitworth steel and metals in which compression had similarly closed up the pores, and by thus increasing their density, increased the resisting power of the metal.

The phenomenon which I have described in earlier papers as exhibited by autographic strain-diagrams and otherwise, the increase of the resistance of the material to distortion, is an effect apparently, of internal molecular changes which do not affect density, and which, in tin and some other metals, result in a *depression of the elastic limit*. If my critic, or any other experimentalist, will study this phenomenon as I have done, he will find, I think, that gun-bronze belongs to what I have called the "tin-class," in which strain produces a *depression* of the elastic limit whenever any effect can be observed at all. I therefore think, the claim of my critic in behalf of the distinguished officer mentioned, can not be sustained.

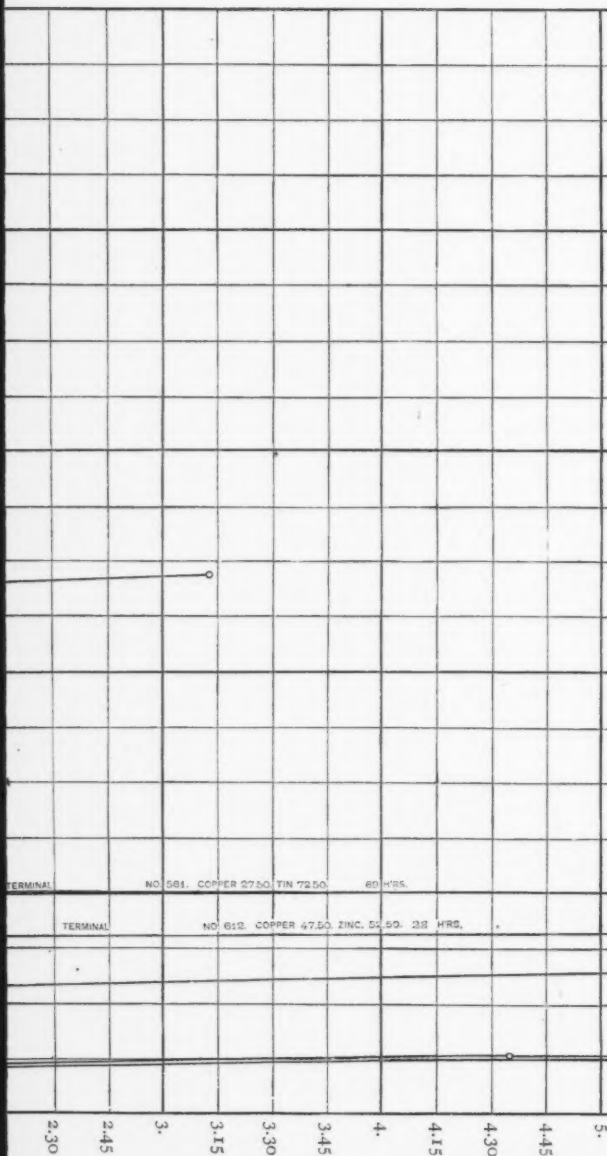
POUNDS.
Decrease of Load.

ROBERT H.



TIME IN TRANSVERSE TESTS OF BARS OF METAL. BARS, 1 INCH SQUARE, 22 INCHES BETWEEN SUPPORTS.

ROBERT H. THURSTON.



A blank graph grid with a vertical axis labeled 4.45, 5, 5.15, 5.30, 5.45 and a horizontal axis labeled NO. 561, NO. 512. A single data point is plotted at approximately (5.30, 5.15).

NO. 612.

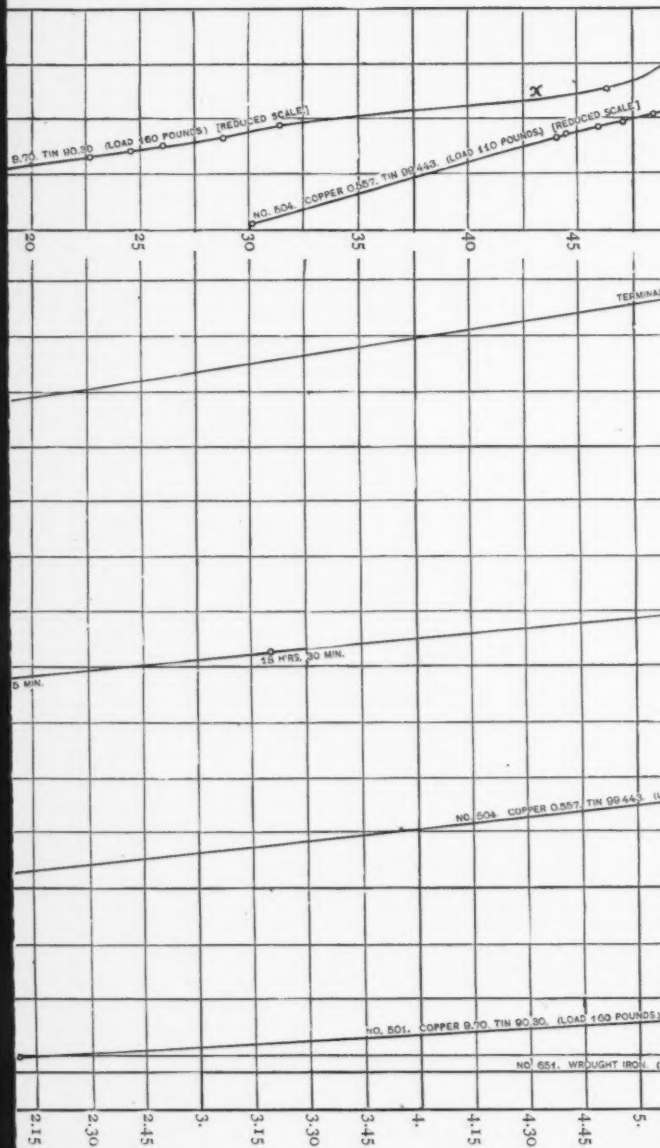
NO. 561.

H. M.
6.

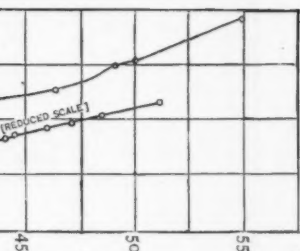
WITH TIME IN TRANSVERSE TESTS OF BARS OF METAL.

BARS, 1 INCH SQUARE 22 INCHES, BETWEEN SUPPORTS.

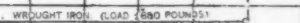
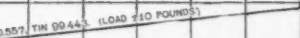
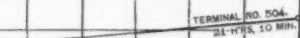
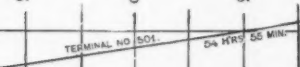
ROBERT H. THURSTON.



METAL.



HOURS.



6.

H. M.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CXXXV.

A MEMOIR OF AMERICAN ENGINEERING.

A Paper by JOHN B. JERVIS, C. E., Honorary Member of the Society.

Read October 18th, 1876.

To the American Society of Civil Engineers :

I have had under consideration, your request to furnish such history of American engineering as could be gathered from my observation and experience. At my age, now past fourscore years, I cannot undertake the labor of looking up documents, so as to be exact in facts, and can only give them approximately as to dates. I necessarily feel embarrassment in undertaking this, from the fact that I have been so personally interested in much that has been done, that my statements may easily be attributed to egotism. In this regard, I hope you will be charitable, and that you will have no occasion to doubt the fidelity in anything I may say.

THE ERIE CANAL.—My first experience was on the original construction of the Erie canal, between Buffalo and Albany, in the State of New York. This work was commenced and ground broken, July, 1817. As the circumstances under which I first entered on this department of work may be interesting, I will occupy a few lines in describing it.

The work had been commenced in the town in which I resided, but the line had not yet been located through a piece of cedar swamp in this vicinity. Hon. Benjamin Wright, the Chief Engineer, resided in the same village—Rome, N. Y. A party of engineers came on to make this

location, but they had no axemen, a portion of the force indispensable for this service. Being well acquainted with my father, Mr. Wright called on him to supply two axemen for this service. My father inquired of me if I would take one of his axemen and do the work. I was then, (1817), near twenty-two years of age, and as the work was expected to take only a few days, I cheerfully assented to go. Mr. N. S. Roberts was the engineer who had charge of the party, a pretty stern sort of a man, and very exacting. Myself and assistant were expert axemen, and with the enthusiasm of a new and untried operation, we entered on the work. It was soon evident our principal was pleased with the manner we executed our duty. My attention was soon directed to the use of the instruments, which at first appeared quite mysterious. In the course of proceeding, it often happened that I was brought to wait a little time with the rodman (as he was called), or target-bearer. At such times, I was led to examine the target and notice the operations. In a day or two, I began to think I could do that duty, and so thoughts rambled in my mind, of learning the art. But then I thought, I had nothing but a common school education, and how should I be able to master the mysteries of such a science as engineering appeared to be? Still I pondered on the subject, and so far concluded, that what others could do, I could do. At the last day of this service, while the party was taking its lunch in a little huddle in the swamp, I ventured, half jest and half earnest, to ask the principal, "What will you give me to go with you, next year" (this was in November), "and carry one of those rods?" To this, he replied that he would give me \$12 per month. This was so prompt that I was a little startled, and began to think very seriously of the matter.

So, you see, I began my experience in engineering as an axeman, and having so well acquitted my duty in this, that it was thought I might succeed in a higher service. With some trepidation, this engagement was settled, and I occupied such evenings and other times as my daily avocations permitted, in the study of surveying, the art at that time regarded as the basis of civil engineering; and on April 10th, 1818, I started from Rome with a locating party consisting of about twelve persons. All but the principal of the party, Mr. Roberts, set off on foot, having a wagon for our baggage and tents. The third day, after a very muddy walk, we reached Geddes, near Syracuse, and pitched our tents. Early in July, we completed the location to the Seneca river, at Montezuma. I was now a regular rodman of three months' experience. The party returned to Rome and was disbanded.

There had been now, a large portion of the middle division, (Utica to Montezuma) put under contract, and several small parties, each with a Resident Engineer, was organized to direct the execution of the work. I was assigned to a party having about 17 miles in charge, in the counties of Madison and Onondaga, under Mr. David S. Bates. The only qualification of my principal was, that he was a land surveyor, of good standing. As my experience in handling levelling instruments was greater than his, he very readily allowed me to run his levels. I do not mention this to his disparagement, but to show the limited range of engineering experience at that time. At the close of the season for work on the canal, I was sent to the quarry to weigh lock-stone for the winter. The next season (1819), my principal was charged with more extended duties, and I was made Resident Engineer of his division, at a salary of \$1.25 per day, and \$0.50 for expenses. This I regarded as satisfactory progress, though I was employed only about nine months of the year.

When this canal was constructed, the State of New York contained about 1 250 000 people. A large part of the State was a wilderness, and the surplus earnings of its citizens were mostly absorbed in clearing the forest, draining the land, and in the erection of necessary houses, offices, mills, and other improvements imperatively called for by a people settling a new country. Of course the financial question was one of profound importance, and to a large portion of the people appeared insuperable. I well remember the alarm of intelligent men, that it would sink the State in irretrievable ruin. But little had been done in such works in this country at that time. The State sought the aid of the general government. One of the Commissioners, Mr. Joshua Foreman, of Onondaga, said to me, that after they had represented at Washington their plan, and made request for aid, Mr. Jefferson said to him—"We are trying to make a canal 3 miles long at this city, and we have not been able to obtain sufficient funds from individuals, the State government and the general government to complete it, and now you ask us to aid you in building a canal 300 miles long through a wilderness. Preposterous!" Failing to obtain aid from other States, or the general government, the State of New York entered single-handed on both these great works, the Erie and Champlain Canals. The financial question, at that time a very serious one, called the attention of the best financiers of the State to its consideration. In this they devised a system, and carried it out with such fiducial integrity that the State received a premium on the 5 per cent. bonds, of 15 per cent.

I well recollect that, among the items of criticism at that day, the question of the skill of the engineer to establish long levels, and as the Rome summit here was 60 miles long, it was confidently predicted that in this case there would be failure. The level was carefully tested by Mr. Canvass White, the Principal Assistant Engineer, and such errors as were found were corrected. When the water was let in, the level proved to be correct. I may remark that, however well the science of levelling is understood, it requires great care to establish a water-level for 60 miles. Previous to this time, some small canals had been made—as the Middlesex in Massachusetts, and some others—to pass falls or rapids in natural navigation. In the State of New York, the Inland Navigation Company constructed, about the year 1798, a canal at the Little Falls of the Mohawk, of about 1 mile in length, with 5 locks, and a canal of 2 miles at Rome, to connect the Mohawk and Wood Creek, and some other improvements by locks and dams. By these improvements the batteau boats, of a capacity of 10 to 15 tons, were able to navigate the natural waters.

Hon. Benjamin Wright, of Rome, N. Y., had been an eminent land surveyor, and with Hon. James Geddes, of Onondaga, and Mr. John Broadhead, of Utica, had made the previous year, (1816) a preliminary survey and estimate of the Erie Canal, and with Mr. G. Lewis Garin, for the Champlain Canal. With the information furnished by these engineers, the State entered on the construction of the work. Mr. Wright was appointed Chief Engineer of the Erie Canal, and Mr. Geddes of the Champlain Canal.

Mr. Wright had Mr. Canvass White as his Principal Assistant, and to him was committed the duty of preparing plans for the mechanical structures. I do not think Mr. Wright made any plans of importance; though he did not draw plans, he was a very sagacious critic of any presented. He excelled in practical judgment. His Principal Assistant delighted in plodding over plans and methods of construction. He prepared the plan for locks, which, considering the times, was highly creditable to his engineering skill.* Mr. White did a very valuable service in discovering the material for hydraulic cement. I have always regarded him as having had the most strict engineering mind of any of his time. The Chief and his Principal Assistant worked remarkably harmonious together, and performed a better service to the canal than either would

* An old and near worn out copy of this lock plan I have preserved; it will be found in the folio of plans of the Delaware & Hudson Canal, now in the Society's rooms.

have done alone. As indicating the temper of the times, I notice, the Chief Engineer called on a carpenter, a Mr. Cady, of Chittenango, to prepare a plan for the wooden trunks of aqueducts. Mr. Cady's plan was adopted, and it will bear the test at this day for that kind of structure.

The Middle Section was essentially completed, at the close of 1819. But there was found more or less deficiencies that required nearly the next season, 1820, to fit it for regular navigation. The duty of attending to the further completion of this Section was intrusted to the writer, which constituted his third year's experience at no advance in salary. In the spring of 1821, I was made Resident Engineer of a Division of the Eastern Section, extending from the Nose to Amsterdam, 17 miles. This was a much more difficult duty than I had on the Middle Section. In several places the bald side hills were washed by the Mohawk river, requiring protection against its floods, and there were many more mechanical structures. This Division and the greater part of the Section between Utica and Schenectady, was completed substantially at the close of 1822. But, as was observed of the Middle Section, there were a number of items still to be done in 1822, and navigation was not opened successfully until September of that year. The work on my Division was mostly done and the accounts settled during the early winter of 1822, and in the spring of 1823, I was assigned the duty of superintending the canal for a section of 50 miles, from the Minden dam to the aqueduct across the Mohawk river. This service was one of valuable experience to me personally. Hitherto I had been *exclusively* engaged in construction, and this gave me opportunity to see the working of the canal in actual operation, and was highly interesting. It moreover gave me experience in the management of such work. The first year, I had not exclusive control, as some of my brother Residents were more or less occupied in completing their work, and I did not have full charge until the month of September. The second year I had entire charge. The Chief Engineer had left for other works, and was seldom on the canal. The Section of the canal under my charge at this time, was about one-seventh of the entire line, and more expensive to maintain than the general average. The work being new, there were frequent failures; but as weak points developed they were repaired, and the work was constantly improved. In many cases it required a good deal of activity to keep up the navigation. This Section was maintained at a cost of \$600 per mile, including a large amount of work in graveling the towing-path. The work was done in a strict business way. Mr. Henry

Seymour was Canal Commissioner. He gave me full authority in regard to everything relating to the work, making an occasional visit and consulting fully on the wants of the work. There were no politics to be cared for. I selected all the foremen, and visited them all twice every week, personally directing them in regard to their work. No other part of the canal exhibited the same economy. In view of my experience in the maintenance of the canal, it appears a strange waste to see the subsequent expenditures of from three to ten times the amount.

In 1825, the canals were completed and opened for navigation by a magnificent celebration.

In March, 1825, I resigned my employment on the Erie canal. I left with the unequivocal compliment of the Canal Commissioner, Mr. Seymour; but I was ambitious to engage on new work, now being projected in many parts of the country. The success of the Erie canal, and the magnitude of its traffic, gave great impulse to canal enterprise. Mr. Wright, the Chief Engineer, had gone to the Chesapeake & Delaware and the James river canals; Mr. White to the Union and Lehigh canals in Pennsylvania, and afterwards to the Delaware & Raritan canal and the Morris canal in New Jersey, the Blackstone in Rhode Island, the Farmington in Connecticut, the Schuylkill, as also the State canals of Pennsylvania, Ohio, and other States. It is hardly necessary now to say some of these canals proved useless as commercial enterprises.

THE DELAWARE & HUDSON CANAL AND RAILWAY was projected by Messrs. Wurtz, of Philadelphia. They owned coal mines in the Lackawanna valley, and entered into this project to obtain a channel for the transportation of their coal to the city of New York, and the valley of the Hudson. Under a charter obtained from Pennsylvania and New York, a company was organized in the city of New York early in March, 1825. A survey of the route, and an estimate of the cost had been made the previous year under the direction of Mr. John L. Sullivan, an engineer from Boston, and Mr. John B. Mills, as his assistant. Mr. Mills conducted the survey. The plan proposed was partly canal and partly improvement of the river by locks and dams. The cost was estimated at \$1 208 000. The actual cost was nearly double this. This was a hardy enterprise for private capital, at that day, and great credit is due the enterprising Board of Directors who persevered through many difficulties to final success. Mr. Philip Howe was the first president, and was succeeded by Mr. John Bolton. The Board of Directors had engaged Mr. Benjamin Wright as Chief Engineer, who requested me to take the posi-

tion of Principal Assistant Engineer, with a view to conducting the details of administration under his general advice. I concluded the engagement March 25th, 1825, and proceeded with Mr. Mills, above referred to, to make an examination of the route. This led me to report against most of the slack-water plan, by dams and locks. The rivers were of a character that did not appear favorable, in my mind, to this kind of improvement. To my views in this report, the Chief Engineer assented, after he had examined the route, and an independent canal was decided on. The size of the canal was 4 feet depth of water, 20 feet width of bottom, and 28 feet width at top water line. The locks were 76 feet long, and 9 feet wide in the chamber, and designed for a boat of 30 tons. The total length of canal from Kingston to Honesdale is 106 miles; it has 110 locks. From the termination of the canal at the forks of the Deyburg (Honesdale), a railroad is constructed 16 miles to the coal mines in Lackawanna valley. Intervening these, was a rise of near 900 feet to the summit of Mosaic mountain, and nearly the same descent to the mines.

At this time, no coal was brought from the anthracite coal fields, except that by arks floated down the Lehigh from Mauch Chunk, nor was the article used in New York.

The canal, commencing at Kingston on the Hudson, traversed the valley of the Rondout and Mamakating, 59 miles to the Delaware river, thence 22 miles up the Delaware to the mouth of the Lackawaxen river, and thence up the same to the forks of the Deyburg at Honesdale. The locks east of the Delaware were made of stone—the face hammered, but not cut—and laid in hydraulic cement, and the backing in quick-lime mortar. This last was a mistake. The locks on the Delaware and Lackawaxen were of dry wall masonry, faced with timber and plank. The canal was mostly completed in autumn of 1828, but not much navigation was attempted until the autumn of 1829, when the first coal was sent down.

The railroad, commencing at the mines at Carbondale, was carried to the summit of the mountain, in about 3 miles. The ascent was made by five incline planes, all but one, on an angle of one-twelfth. A less number of planes would have been chosen if the formation of the country had been favorable. The planes were operated by stationary steam engines. The cars were moved by endless chains passing round on sheave wheels, at the head and foot. In order to guard against the slipping of the chain on the upper sheave, steel studs were sunken in its groove. When

put in operation the machinery worked well, but the chain frequently broke, and had to be abandoned; hempen rope was substituted instead. It was apprehended the rope would not have sufficient hold in the sheave to prevent slipping. To obviate this, a second sheave was placed on the same shaft, by means of which the rope had two holds in the groove instead of one, which was found to be quite sufficient, and the whole worked satisfactorily. Subsequently the hempen rope gave place to one made of wire.

After reaching the summit, the line passed about $1\frac{1}{2}$ miles to the brow of the mountain on the opposite side, and then had a descent of near 500 feet in about one mile. To provide means to resist the preponderating gravity of loaded trains as compared with empty trains, was a serious matter. The method described, in European engineering, was by a friction brake; to this, there were considered several objections, when applied to so large a preponderating force. It occurred to the writer that it might be possible to make use of the resistance of the atmosphere to secure the object. It was found that experiments had been made on this resistance, which so far as records could be found, were on a surface of one square foot, and it was deemed advisable to have them made on a larger area. Accordingly, I had an apparatus prepared, in which I could use a surface from 4 to 21 feet area. With this, I made a great number of experiments at different velocities, from which it appeared that the resistance was rather greater than in those referred to; whence I decided that air was the best element for controlling the preponderating force, and immediately set to work to devise the method of application.

The plan adopted was to place a large spur wheel on the upright shaft, immediately under the sheave wheel; the spur, extending to each side of the road, worked the pinions of these shafts, and as the sheave wheel revolved, motion was given to the sails, which from their natural position, drove the air in opposite directions, and so prevented gyration. Each shaft had four sails made of thin boards, of about 20 square feet area. It will be noticed this was a very simple sort of machine. On one of the sail shafts was a powerful friction brake, to be used in stopping the trains at the head and foot of the planes, and for any emergency that might require it. This I called a pneumatic convoy. It will be readily apprehended that I felt no little anxiety in the success of the experiment as the first train was passed over the angle of the plane. Of course, the friction brake was at command, though its use was not required. The train moved off, the sails moved regularly and held it throughout to an

uniform velocity of about 4 miles per hour, as had been calculated; the result was complete. As business increased on the road, it became necessary to increase the speed of trains, and to reduce the area of the sails, and I believe, finally only their arms were left.

This plane was about $\frac{3}{4}$ mile long, and had a descent of about 350 feet. There were two other planes of this kind (called "self-acting" planes), to provide in part for the descent to the canal. The second was a short distance from the first, and thence the railway was carried on a grade of 1 to 120, about 6 miles, to the head of the third plane. On this grade, the loaded trains descended by gravitation. From the third self-acting plane to the head of the canal (about 3 miles), the road was constructed on a grade of 1 to 200. On this and also on the 6 mile grade, it was intended to use locomotive power—on the latter to haul back the empty cars. But this feature was abandoned, though the engines had been obtained. It was supposed the road would carry an engine of from 4 to 5 tons; they were made about one-half heavier. After repeated trials, it was decided the road was not sufficiently strong to sustain them with safety, and horses and mules were substituted in their place.

Now, this railroad has been materially changed. I left Mr. James Archbald, one of my Resident Engineers, in charge as Superintendent of the railway and the mines—that is, I recommended him, and the company gave him the superintendency. He had an excellent engineering mind, of great practical sagacity, and was eminently upright in purpose. After a few years he designed and executed an important improvement. In this, he abandoned plane No. 8 (the last self-acting plane), and carried the 6 mile-grade to the head of the canal at Honesdale. He then made a new plane worked by steam power, at the head of the canal, up which he hauled the empty cars to a certain height; and also a new railway on a grade that would allow them to descend by gravitation, as far as the country would permit; then by another plane of same kind, the cars were hauled up to such height as could be commanded, and a gravity railway was made as far as the ground would permit. So alternatively, by planes worked by steam and by gravitation, he completed a return railway from the canal to plane No. 7 (the second self-acting plane), a distance of about 10 miles. This dispensed with all moving power, and completed the machinery character of the railway. It is now worked by stationary steam and gravitation, with no moving power of importance.

Subsequently, Mr. Archbald was employed as the Chief Engineer of the Pennsylvania Coal Company, for which he constructed a railway from its mines at Pittston, through Cabb's gap and the valley of the Wallenpaupac to Hawley, a town on the Delaware & Hudson canal, about 8 miles from the head at Honesdale. This railway is about 40 miles long and wholly a stationary steam and gravitation railway, in both directions; it has worked in the most satisfactory manner. It is obvious the above plan would only be useful when there was great disparity between the elevations of the valleys and adjacent hills. This was peculiarly the condition in these cases, and transportation was made at moderate expense over the mountainous country.

To return to the canal. Mr. Wright resigned the position of Chief Engineer about two years after the commencement of the work, and the writer was appointed to that position. In the autumn of 1829, there was some navigation on the canal, and a few hundred tons of coal were transported to tidewater. There were many impediments to the progress of navigation, but during 1830, there was considerable traffic, the work steadily improved and the quantity of coal delivered at tidewater was increased. I recommended Mr. Russell T. Lord, who had been a Resident Engineer on the canal, as General Superintendent. He was a man of excellent executive ability, and conducted the administration with great good sense and fidelity to his duty. The canal business increased, and after a few years, Mr. Lord made an improvement by increasing the depth of water, so that he raised the tonnage of the boats from 30 to 50 tons, increasing the capacity of the canal and the economy of transportation. In this, he only altered the locks, so as to meet the rise in water, leaving the chamber the same in width and length. The advantage that had by this means been realized, led to a more radical enlargement, by which the locks were rebuilt with chambers 90 feet by 15, instead of 76 by 9. The depth of water was raised to 6 feet, and the canal more or less widened. This would all have been well, had the widening corresponded, but it was not sufficient to give the proper ratio between the section of the boat and the section of the canal. The result was, it did not improve the economy of transportation; it however, increased the tonnage capacity of the canal, which was an ample compensation for the cost of the improvement. Mr. Lord replaced two, and added other two aqueducts by which the canal was carried over streams, supported by wire cables. The traffic of the canal, (mostly coal) increased much beyond

the original anticipation ; it is now carrying annually about 1 250 000 tons of coal, and is in a highly prosperous condition.

The original works cost about \$2 500 000. This sum put in operation, 106 miles of canal of generally difficult construction, with 110 locks, and 16 miles of railway with an elevation of about 1 900 feet. The company obtained the credit of the State for a loan of \$800 000, which was all punctually refunded.*

I must not forget to notice the aid I received in arranging the details of machinery, from Mr. John H. McAlpine (the father of Mr. William J. McAlpine†). Mr. McAlpine was an accomplished machinist, to whose superintendence I intrusted the machinery department, in which he performed his duties in a very satisfactory manner. While in this service, Mr. McAlpine introduced to me his son William, and requested that I should put him in one of the engineering parties. The son was then about sixteen years old, a light, but active and pleasant boy. Advancing from station to station, he was many years in my employ ; manifesting capacity, industry and fidelity, he was one of my most esteemed assistants. His subsequent career and standing is before the world, as among the eminent engineers of the country. He is still in the vigor of manhood, and can tell his own story.

THE MOHAWK & HUDSON RAILWAY.—In May, 1830, I left the service of the Delaware & Hudson Canal Company, except that I was to make occasional visits to the work for the then ensuing year, and accepted the appointment of Chief Engineer of the Mohawk & Hudson Railway. This line extended from Albany on the Hudson to Schenectady on the Mohawk. Its main object was the transportation of passengers. At that time, passenger boats on the Erie canal brought passengers from Buffalo to Schenectady, and together with the stage coaches, transported a large number.

The intermediate country was a table land of a fair character for a good line and easy grades ; but that was reached by a sudden rise of near 200 feet from the Hudson, and over 100 feet from the Mohawk. These rises were at that time regarded as requiring inclined planes, worked by stationary engines. Consequently, this railway was constructed with an inclined plane at each end. The adhesive power, the means of transmitting power by locomotive engines, was then regarded much

* I send to the Society, a number of the plans of structures on the Delaware & Hudson Canal and the Carbondale Railroad, which I have preserved.

† Member and Past President of the Society.

below what it was subsequently found to be. After working with the inclined planes a few years, their inconvenience for a miscellaneous traffic was demonstrated, and a commission appointed to ascertain the practicability of dispensing with them. I was a member of the commission, and examined the engineering view of the question. We found a line rising from Albany at a grade of 80 feet per mile, and from Schenectady of 40 feet per mile, which the commissioners recommended; the line was accordingly changed in this respect, and has subsequently been worked by locomotive steam power.

This railway was constructed with a double track. It had the ordinary features of railways at that time constructed in this country—namely a wooden rail capped with a plate of iron, which in the excavations was laid on stone blocks, and on embankments, with timber cross-ties. While in use, the inclined planes worked very smoothly, and I believe, no serious accident occurred.

A locomotive engine was made for this line by the West Point Foundry Association. It weighed between 4 and 5 tons, had 4 wrought iron wheels, and carried from 75 to 125 passengers in a train, at a speed of 25 miles per hour. A second locomotive was obtained from the works of Robert Stevenson, England, which weighed about 7 tons, and was much more powerful than the other. The first was named De Witt Clinton, and the second, John Bull. The action of the John Bull was observed to be rather severe on this kind of rail. It being placed on 4 wheels, the overhanging caused a sharp and disagreeable motion of the engine. This circumstance, with others that came under my observation, induced me to seek some remedy for the weight, and to secure a more steady motion. The idea of more wheels was natural for the first object, but to spread the base was supposed unfavorable for curves; and I was finally led to the plan of a 4 wheeled truck under the forward part of the engine, as a support for that end, and as a guide. By this I had 4, instead of 2, wheels and could place the trucks so far from the drivers as to cause less vibrating motion. A new engine was built by the West Point Foundry Association on this plan, and the truck was found to work satisfactorily. I then made a new plan for an engine for the Schenectady & Saratoga Railway (of which I was the Chief Engineer), and sent it to Mr. Stevenson, who built the engine. This engine weighed about 6 tons; it moved with perfect beauty, and I may be pardoned, I think, in saying that my first ride on it was a great delight. The smooth action of its parts, and the steadiness of its motion gave me great satisfaction. A

year or two later, the railway was extended westward, and from time to time, continued, until it reached the Pacific Ocean at San Francisco. On the entire route, the truck arrangement was considered as a complete success, and is now the general plan of American locomotives. The truck itself was not a new idea with me ; it had in principle been used, but was very generally supposed to be impracticable for high speed.

The Mohawk & Hudson and the Schenectady & Saratoga Railways were completed, and in April, 1833, I engaged with the Canal Commissioners of New York, as Chief Engineer for the Chenango canal.

THE CHENANGO CANAL is about 98 miles long and has about 100 locks. The cost was about \$2 000 000; it extended from Utica on the Mohawk river, through the Oriskany and Chenango valleys to Binghamton on the Susquehanna. The route was moderately favorable. The most distinctive feature, is the resort to artificial reservoirs to supply its summit with water. At the day this canal was commenced, it is believed no such dependence was relied on, to supply canals in this country. Such means had been adopted to supply canals in Europe, and it had been held that one-third of the rain-fall could be depended on, over the loss by absorption and evaporation. As this ratio would vary in different situations, I decided to establish a rain gauge and a sluice for measuring the water that flowed from the valley. On one of the valleys, this was attended to daily for one year, and on another valley, from June to December. Mr. William J. McAlpine was the Resident Engineer on the Summit Section and had charge of the rain and sluice gauges. The matter was diligently attended to, and the result from these gauges was to establish 40 per cent. as the proportion of rainfall which on an average soil could be secured in a reservoir.*

During the time of my engagement as the Chief Engineer of the Chenango canal, the traffic on the Erie canal had been so large, that measures were in contemplation to double the locks and to enlarge the canal.

ERIE CANAL ENLARGEMENT.—The Canal Board had decided to enlarge the section to 5 feet depth and 50 feet width. This decision was not fully approved as being adequate to the wants of the future traffic, and the matter was discussed by the public, which caused indecision and suspended operations. My connection with Commissioner Bouck on the Chenango canal, led to much conversation on this matter, and at his re-

* The plan for one of these reservoirs as also for other structures on this canal will be found in the folio, labelled Chenango Canal and Erie Canal Enlargement, in the Society's Rooms.

quest, I investigated the question of the ratio the sections of boats should bear to the section of canal, deducing the comparative economy of transportation. Without dwelling on preliminary facts, I may state these discussions led the Canal Board to order a careful survey of the work, and estimates to be prepared on the basis of enlargement to a depth of 6 and also 7 feet of water, the widths being respectively 60 and 70 feet.

In accordance with the above, engineers were appointed to make the examinations on the respective sections as directed by the Commissioners, and to the writer the Eastern Section was assigned. The surveys and estimates were made in 1835. The surveys of this section were made by Mr. William J. McAlpine, who at that time was one of the Resident Engineers on the Chenango canal. Though I was in charge as Chief Engineer of this canal, I gave a large share of personal attention to the question of enlargement. It had appeared to me that errors had been made in the original work which it was important to correct, as far as practicable, in the enlargement. At a point about 8 miles above Albany, commenced the location of what was termed the "nine locks." In a distance of about 2 miles there were, I think, 17 locks. At one cluster the 9, and at another the 4 locks, had much contracted pond reaches, which was found to be very unfavorable to the large traffic that then existed. I directed Mr. McAlpine to commence a survey at a point about 1 mile below the 9 locks, to run the level above the 4 locks, and ascertain if it was practicable to find ground on which these locks could be located so as to give a pretty uniform space for the pond reaches between them. The ground was pretty rough and rocky, but a satisfactory location was found, that allowed the locks to be very evenly distributed on a new line about $3\frac{1}{2}$ miles in length. I therefore recommended the Commissioners to abandon the old for the new line; this was done and the new line constructed, which has proved a very satisfactory improvement.

A short distance beyond the point above referred to, the canal line crossed the Mohawk river to the north side, was continued on that side 12 miles, and then crossed back to the south side of the river. This involved two aqueducts over the river, averaging each, near 1000 feet in length. I had a line continued on the south side of the river, with a view to judge of the propriety of dispensing with the aqueducts. The surveys and estimates induced me to recommend the abandonment of the canal on the north side, and of course the two aqueducts, and to adopt the new line on the south side of the river. On this question the

Canal Board was equally divided, and of course, the recommendation was not adopted.

The next point of importance in changing the original plan, was on a section from Schoharrie creek, to a point, 4 miles above. On the west bank of Schoharrie creek, the canal was locked into it, and by means of a dam, a pool was made as the channel of the canal across the creek. At the point 4 miles above, was another lock, and about midway on this section, a pretty large creek, though much smaller than the Schoharrie, crossed the canal by means of a dam. These were turbulent streams, and often the source of much vexatious trouble to the navigation. I proposed to take up the 2 lift locks, by which the level of the canal was raised sufficient to cross over both the streams by aqueducts. This change the Canal Board sanctioned, and it has been a great benefit to the navigation.

There were other streams crossed by means of dams for which I arranged the levels, so as to be crossed by aqueducts. This method provided for putting small streams under the canal by means of culverts, many of which from the low level had been allowed to flow into the canal. At the Little Falls of the Mohawk, I wholly rearranged the flight of locks.

The surveys on the canal west of my section were made by Messrs. Nathan Roberts, F. C. Mills, and Holmes Hutchinson. The instructions of the Canal Board did not propose nor ask the several engineers to express opinions as to what size they regarded most appropriate, but Mr. Roberts and Mr. Hutchinson had given their opinion in submitting their report. On discovering this, Mr. Mills and myself made a special report on this point. We declared in favor of 8 feet depth and 80 feet width, which was the size I had regarded most suitable. The Canal Board, however, decided on 70 feet width and 7 feet deep, and it has been so constructed. The plan of stone arches for the towing-path on the aqueducts, when the height required a timber trunk for the boat channel, was proposed by me at the same time, and has been adopted in the subsequent works on the canal.

Early in the year 1836, the Canal Board decided to proceed with the improvement as above stated, and on the Eastern Division to make double locks. Plans and specifications of the work were proposed, and contracts made early in the summer of 1836, and the work commenced. With Mr. William J. McAlpine as Principal Assistant, I was appointed Chief Engineer of the Eastern Division.

No one appreciated more than I, the importance of substantive and durable works, but unfortunately at that time, there was so high an estimate of the value of the canal, that the ideas of men were extravagant, and advocated work of an expensive character, that was in no way more substantial or useful. This induced a more or less expensive policy, that increased the cost of the work much beyond what was necessary. I had recommended, that the chambers of locks for a canal 7 feet deep by 70 feet top width, should be 16 feet wide and 115 feet between the gates. The Canal Board, on the petition of navigators, increased the width to 18 feet, which I regarded a decided error. A navigation with few boats may be conducted on a comparatively narrow channel; but for a large traffic, there is a proper relation between the area of boat and area of channel that secures the best economy in transportation. The boatman will make the boat as wide as he can pass the locks, with simply room to pass on the open canal. They analyze nothing, but suppose the acme of economy is in the largest possible load they can carry; very much on the theory of most railway superintendents who consider the largest possible train as securing the best economy in transportation, with no more attempt at scientific analysis than that of the boatman. It is now the opinion of the most intelligent navigators of the canal, that the locks are too wide for the best economy of transportation.

THE CROTON AQUEDUCT.—In September, 1836, a committee of the Croton Aqueduct called on me with a proposition that I should accept the position of Chief Engineer of that work, and in October following, I accepted the position. The Canal Commissioner (Hon. William C. Bouck), with whom I was especially associated, was reluctant to have me leave the canal, but said he would not object to my accepting the charge of the Aqueduct if I thought it my interest to do so. I have been charged with intriguing for the engineership of this work, but that is an entire mistake, as I had not mentioned the subject to any one, and consequently was quite surprised at receiving the proposition. All the intercourse I had had on the subject, was an answer to a letter of the Chairman of the Croton, Aqueduct Board, requesting me to send him any copies I had of the forms of contract and specifications of the State canals, in which there was not the least allusion to my having anything to do with the work. At first I declined the offer, not feeling willing to interfere with Major Douglass, who, I knew, had a high estimate of the professional importance of the work. But the committee placed the matter on the distinct ground that they had decided a change was necessary, that I was preferred, and

if I did not accept they should seek some one else, and they wished me to consider Major Douglass as out of the question. Under such circumstances, I saw no impropriety in accepting a position that appeared professionally desirable, and offered without the least effort or knowledge on my part. Accordingly I resigned my position on the Erie Canal Enlargement, and Mr. McAlpine, my Principal Assistant, was appointed to succeed me.

Surveys had been commenced in 1833, with a view to examine the feasibility of introducing the waters of the Croton river into the city of New York. Major D. B. Douglass was occupied during the seasons of 1833-34, and made estimates and reports of his surveys. Mr. John Martineau was engaged on the same in 1834. They made independent surveys and reports to the Board of Water Commissioners, and substantially agreed in recommending the Croton as the best source for supplying New York with water. In neither case, was there much presented as to definite plans of the works, by which the object was to be secured; these were left in general form, for future elaboration.

In the spring of 1835, the city of New York accepted the plan and authorized the Board of Water Commissioners to proceed with the construction of the work. Messrs. Stephen Allen, Saul Alley, Charles Dusenberry, Benjamin M. Brown, and William W. Fox constituted the Board. They appointed Major Douglass, Chief Engineer, and instructed him to proceed to locate the line and prepare plans and specifications for the work. He made the location of the line of aqueduct from the Croton river to the North bank of the Harlem river, 33 miles; and determined the grade of the aqueduct at about 13½ inches to the mile. It was in the main, well located.

In regard to plans of work, he proposed a cross section of the masonry of the conduit, which with some modification was adopted. So far as I have known, he prepared no specifications for the work that were approved by the Commissioners. I am thus particular, for the reason that Major Douglass' friends have claimed for him the credit of preparing the plans on which the work was constructed. In addition to the above, some plans of bridges and culverts were prepared by him, but none of them were adopted. He had been occupied on the work as Chief Engineer about one and a half years after it was authorized. In whatever follows,* the plans and specifications of the several works were prepared

* I shall not go into particulars in relation to the plans, except it may be, in regard to some peculiarities, as I have prepared a folio of the drawings and of specifications of the manner of work, which is at the rooms of the Society, and to which I refer.

by the writer exclusively. I proceed to give some particulars of the more important plans of the work.

The Dam across the Croton River was proposed for the head of the aqueduct, and required to be above the surface of the river, 40 feet, to meet the grade of the aqueduct at its head. The south side of the river was a rock, rising from the water at about 1 to 1½ to the head of the aqueduct. The dam and its abutments on the river side were built on rock, and the extension of the weir was made by excavating the rock into the hill. But I need not take space for this part of the work. The length of the weir was too short for the flood that came just before the dam was completed, and the embankment that connected the dam with the north shore of the river gave away. The masonry of the dam was not at all disturbed by the flood, though the water rose 12 feet above the level of the overfall or lip of the dam. The distance from the abutment to the hill on the north side of the river, was about 200 feet. I had hoped, by confining the dam to the rock on the South side, to secure the benefit of the rock to break the fall of water in times of flood, which would have been very well had the length of weir been sufficient. The result showed this to be an error; I then recommended that the dam be carried entirely across from the old abutment on the south side to the high grounds on the north side, making an extension to the weir of the dam, of 180 feet.

There was no rock for a foundation at this part, and all the works required to be artificial. The great point in such cases, is the means of counteracting the fall of a large body of water, over artificial works 40 feet high. The plans show this to be done, by giving a curved form to the lower side of the dam, by which the course of falling water is gradually changed from a vertical to a horizontal direction, when it reaches the apron and the reaction of the water in the river below. The curve in this case has worked very satisfactorily, the water in a high or low stage, following close on the face of the masonry. There was no reasonable way of turning the course of the river during the erection of the work, and no means to provide for the water except the waste culvert in the old dam, which was about 6 feet above the surface of the river. The ordinary width of the river was about 100 feet, and the greatest depth of water, 15 feet. The culvert above mentioned and temporary sluices over, were the only means for protecting the work, in floods that occasionally occurred in the river. It will readily be realized that for the long time required,

this was a work involving great anxiety, in any prospect of rain. The first season, the work was carried up about half its height, and then covered by a plank and timber aproning, to protect it from the winter and spring floods. Through the kind providence of God, the storm was so tempered that the work proceeded until completed, with little interruption, except that of the winter.

Though varied methods were adopted in preparing the foundation, and bringing the work up to the surface level of the river, there has occurred no settlement to mar the work or materially to affect its level. So far as I know, the dam is in no essential respect impaired after 33 years' use, though at one time a flood rose to the height of 8 feet on the crown or lip of the dam.

General Work of the Croton Aqueduct.—The method of constructing the conduit, the form of ground at different places, the ventilators, the waste weirs, culverts and the foundation walls, are explained by the plans, the form of contracts, and the specifications sent herewith.

Sing Sing Bridge.—The only peculiarity it is necessary to notice, is the plan of cast-iron lining for the conduit. This was introduced in order to guard against leakage that by possibility might percolate through the masonry. There was good reason to suppose such would be very small, and of no importance as to waste of water; but in this climate, leakage amounting to only a sweating of the arch stone in the bridge masonry, would tend to disintegrate the most durable stone, and it was therefore important to arrest it by every means practicable. This iron lining appears in the cross-section of the conduit over the bridge. The method has been in the main successful, though it requires some attention to correct the effect of longitudinal contraction and expansion. I would recommend for such cases, an iron pipe, put together with faucet and spigot joint, as a more perfect method. Of course, it would require a large pipe to carry the water of the conduit without extra fall, but it is practicable to cast pipe of sufficient size for the Croton conduit.

The centering of the Sing Sing arch was on the plan adopted for the Waterloo bridge at London, and is an excellent one for such arches. As an evidence of the close joints of the masonry of this arch, I had a level taken before striking the centres, and found the settlement of the arch was less than half an inch. The work in this arch has no superior, in comparison with other arches of this size, and of which this item is recorded. The span is 88 feet.

The Harlem River Bridge is about one quarter of a mile in length, and about one-half or 600 feet, is across the Harlem river. The plan shows the bridge to be about 114 feet above tide water. The bed of the river was a deep mud, interspersed with large boulder stone, mostly from 500 pounds to 2 tons, and some of much greater weight. The subsoil under the mud was in some places, rock in place, and at other parts a clean sand. We found no boulders of any amount after striking the sand. The depth of water in the channel was about 20 feet. In the deep part of the channel there was only very little mud. The boulders mentioned above, were very troublesome to the work of driving the sheeting timbers of the coffer dam. After the coffers were emptied, many of the sheeting piles were found broken into shivers, which caused much trouble in managing the coffers. In the line of the coffer sheeting, many of the boulders were "lewisd" and hauled out from 10 to 15 feet below the surface of the river. Boulders weighing 15 tons were so removed, but those left, gave the difficulty referred to.

The great point in supporting a bridge of this height, was to so prepare the foundations, that there would be no unequal settlement in the masonry. As some of the piers were on solid rock, and others on pile foundations, it was a subject of much anxiety. For the central pier that came in the channel, the excavation within the coffer dam was carried down, nearly 40 feet, without finding any material but sand. A boring in this coffer for a depth of 50 feet showed only sand. Piles, 12 inches in diameter, were then driven, mostly 35, and some 40 feet in length. It will be noticed, these piles were driven to a depth of 80 feet in some cases below the surface of the river, or 194 feet below the parapet of the bridge. A course of white oak timber was fitted on the heads of each row of piles, and the space between, filled with concrete masonry. Then a second course of white oak timber was put across, on the first course over the piles, and the spaces filled as before. On this, the masonry of the pier was commenced. The foundation was made much larger than the shaft of the pier, with offsets to about the surface of the river. In all the pile foundations, it was meant to give such breadth as would be a protection against settlement.

The bond in the courses of masonry in the piers, was thoroughly secured by preparing a plan for each course, and requiring every stone to conform to the plan. It will be noticed, the shafts of the piers are hollow, which was not new, as it had been practised in European bridges.

This bridge has now stood about thirty years, and I have not known of any failure.

The plans I present, will show the arrangement for pipes. It was made for two 4-foot pipes, this being equivalent to the capacity of the masonry conduit. As a measure of economy, and a plan that would provide for the wants of the city for many years, the Water Commissioners decided that 3-foot pipe should be put down, to be replaced with 4-foot pipe when necessary. I preferred two pipes, as affording the facility for repairing or removing one, while the other would supply the city. The gate houses were arranged for 4-foot pipe, and all that was necessary was to take up one line of 3-foot pipe and put down one of 4 feet, as more water was wanted. As subsequent events have shown, it would have been the best economy to have put the 4-foot pipe down, in the original construction; for in some way, the management did not take up either of the 3-foot pipes when more water was wanted, but placed a wrought iron pipe over them, raising the parapets of the bridge, and incurring a much larger expense than would have been required by the 4-foot pipes in the outset. I protested against this, as marring the simplicity, economy, and usefulness of the original plan, but I had not then any official charge, and my protest was unheeded.

The plans referred to,* show the details and method of the pipe works across Manhattan valley, and description does not appear necessary. The same may be said of Clendinning bridge, and of the receiving reservoir.

The Distributing Reservoir.—The leading feature of this, is the plan of hollow walls. The retaining walls were required to sustain a pressure of 40 feet of water. The natural surface of the ground was about the level of the bottom water line of the reservoir. A solid retaining wall of this height would require to be very thick. A single thick wall has the objection that it requires more time to ripen, or become well set in the cement, and further, the cost of such a wall will be greater for the same support than a wall having a broader base and less aggregate material. In this view, I prepared the plan of a hollow wall, that is, two parallel walls connected by cross walls, and the cross walls connected at the top by brick arches. This kind of work was carried up within about 12 feet of the top or coping of the reservoir, and then a single wall completed the work. An

* For a list of the plans referred to, here and elsewhere in this paper, see March Proceedings, (Vol. III.) "Additions to Library and Museum."

opening was left in the cross walls so that a man may go round the whole and discover any leak there may be in the masonry. The working of this plan has proved very satisfactory.

I send herewith a description of the Croton Aqueduct, written in 1842. Some slight modifications occurred in the unfinished works after this was written, but nothing of essential importance.

COCHITUATE OR BOSTON AQUEDUCT.—The city of Boston had made an unsuccessful effort to procure a supply of water. The difficulty arose from a divided opinion as to which of several projects was the best. A Board of Commissioners had reported the Cochituate as the best of the several projects, but the opposition of the friends of other projects was so strong, that the measure was defeated by a vote of the citizens, in the spring of 1845. Not willing to abandon an enterprise so important to the interests of the city, the City Council appointed a committee to institute necessary measures to secure a supply of water. This committee decided to select a commission, consisting of one person from the city of New York, and one from Philadelphia, to make an investigation of all the projects that had been presented, and report such as they should find to be the best.

For this, they engaged Mr. Walter Johnson, of Philadelphia, (a Professor in a scientific institution,) and the writer of this Memoir. I went to Boston, and there met Mr. Johnson, and the members of the Water Committee.

After making some general examination, it was apparent to me that the duty was essentially a matter of engineering, and that under the circumstances existing, it would demand very careful examination and great prudence to bring out successfully the varied features of the proposition. Not having any previous personal acquaintance with my associate, with the fact that he was not professionally an engineer, I concluded not to enter upon the service, unless the engineering was placed entirely in my hands. This decision was not favorably received by the Committee or Mr. Johnson, but I thought "one poor general was better in command than two good ones," so I adhered. The Committee urged that it wanted the moral force of two commissioners; "very well," I said, "the report may be made in the plural, and I shall have no objection to Mr. Johnson signing it with me, and he may be charged with certain duties that will not conflict with the general engineering." To this the committee assented, and the investigation proceeded.

I engaged Mr. Henry Tracy, a civil engineer of experience and ability, to take charge of the surveys and estimates, a service he performed with prudence and ability. In about five months, the examinations and report were completed, by the recommendation of the Coehituate as the best source.

As a suggestion, that may be useful to the younger members of the profession—I kept strictly my own opinion until it was tested by the result of a full examination; not even my confidential assistant ever obtained from me, the least expression of my views. This was not because I was afraid to trust him, but that he might say at all times, he did not know my opinion. In such a case, there is great anxiety with the different parties to obtain knowledge of the prospect of their respective projects, and the least intimation would inevitably lead to discussion. At one time, the Committee was impatient to get my views. My associate had been rather leaky, but he was unable to quote me, and the Committee appeared to think I should at least give them an intimation. I replied, they would know my opinion when they had it in writing at the end of a report. They were not satisfied, and seemed to think I had not treated them with proper deference. In the final result, they expressed themselves as well satisfied with the course I had pursued. The report of the Commissioners was generally satisfactory, and the work was authorized to be constructed.

The work for the aqueduct was commenced in the spring of 1826. The Board of Water Commissioners was composed of three members, Messrs. Nathan Hale, James Baldwin, and T. B. Curtiss; Mr. E. S. Chesbrough, was appointed Engineer of the conduit and its appurtenances, and Mr. William E. Whitwell, of the piping in the city. The writer was engaged as Consulting Engineer, and continued to the close of 1848, when the works were completed, and the water introduced in the city, with enthusiastic demonstrations by the citizens. Any further description I properly leave for the acting engineers, who had charge of the construction of the work.

THE HUDSON RIVER RAILWAY.—Some surveys and estimates had been made for this enterprise, mostly by the aid and influence of Mr. James Boorman, a wealthy and spirited merchant of New York. The project, however, did not obtain much favor with the public. It was generally regarded as unpromising, in view of the competition of the excellent navigation of the Hudson river. In the summer of 1845, Mr.

Boorman called on me, to enlist my services in the examination of the project. I had a favorable opinion of the enterprise, but well understood the difficulty in convincing capitalists that it would be a commercial success. The general character of the country for such a work clearly indicated that it would be expensive, and the competition it must meet from a steamboat navigation conducted on a magnificent scale, led the public to regard it as an extremely hazardous, if not a hopeless enterprise. After considering the proposition of Mr. Boorman, I decided to engage upon the work, and during the ensuing autumn employed Mr. Henry Tracy to make such surveys as the season and limited funds would permit.

Having given my attention to the project, I devoted much time to gathering and analyzing statistics, mostly as to passenger traffic. Mr. Tracy conducted the surveys with great skill, and for the limited means he had, obtained important information. His services were directed to the section between New York and Fishkill, about 60 miles. For the balance of the line, the survey of Mr. Morgan—which left the river at Fishkill, and pursued an inland route—and his report, was the basis of the report I made to a company of citizens in January, 1846.

Under this report, a charter for a company was obtained and the corporation organized. Generally the public mind was sceptical, and subscriptions to the capital stock progressed slowly. The indomitable energy of Mr. Boorman succeeded in raising the amount to \$3 000 000. Loans were depended on, for the balance required.

From the general want of confidence in the undertaking, the organization was not perfected until late the following winter. In the spring of 1847, the Board of Directors appointed the writer, Chief Engineer of the railway, and proceedings were had to commence the work soon after.

As surveys for actual location proceeded, it was soon evident there were obstacles to be encountered that had been very imperfectly developed by the limited character of the original surveys. The rock cuttings in the Highland districts proved to be unusually difficult of excavation. Very little earth could be had in this district for the fillings, and to make these of rock cuttings, especially in those places where the fillings sunk to a great depth in the mud, was very expensive. The long line of exposure to the surf of the river was a serious matter. The great difficulties of the work lay between New York and Poughkeepsie. Above

the latter place, the rock was of a slaty character and easily excavated, and the shore had less exposure to the action of the river.

After a more careful examination of the inland route, and also by a survey of the river route made by Mr. John T. Clark, an Assistant Engineer, I made a report to the Board of Directors in January, 1848. In this, I recommended the adoption of the river route for the whole line. This report was approved, and the work has been constructed on the river route. It was open for transportation to Poughkeepsie about the close of 1849. For the most part, this section of the railway was graded for a double track, but only a single track was laid at this time. No work of construction had been done above Poughkeepsie.

In August, 1849, I resigned the position of Chief Engineer, and Mr. W. C. Young was appointed in my place. I continued, however, as Consulting Engineer until the spring of 1850. At this time, I found that my views in regard to the management and operation of the railway did not harmonize with those of Mr. Boorman, and I therefore resigned and have had no connection with it since then.

It is impossible at this day, for any one not engaged in the persistent efforts of the time, to appreciate the hardy character of this enterprise. The financial and engineering difficulties were very great. In regard to the former, I can hardly conceive that more could have been done than was effected by Mr. Boorman and a very respectable Board of Directors. In regard to the latter, the natural impediments were rendered especially severe by difficulties of finance. These in the face of competition from steamboats of great magnificence effecting very cheap transportation, and the incredulity of the public mind as to the capacity of the railway to maintain such competition, rendered the prosecution of the work extremely embarrassing.

The landed proprietors along the route were in general very hostile to the enterprise, declaring it would destroy the natural beauty of the country as well as fail in its commercial object. On this point I had claimed that the natural scenery would be improved; the shores washed by the river would be protected by the walls of the railway, and the trees, no longer undermined and thrown down by the river surf, would grow more beautiful; and that the railway thus combining works of art with those of nature would improve the scenery. For this I was freely reproached as scarcely less than a barbarian. As to my very modest statement, that the railway trains would be able to travel at the rate of

30 miles per hour, I was called to account in some very decided poetic effusions.

I wrote an article—published in *Hunt's Merchants' Magazine*, in November, 1846—in which, discussing the subject generally, I took the ground that not only along the Hudson, but also on other steamboat routes, the railway would be a successful competitor. In this article, speaking of railways, I said—"The system is viewed as one that mocks the age, its progress has startled the most cautious, its developments are revolutionizing the social and commercial affairs of mankind." This was criticised as a very presumptuous statement. I notice these matters as curiosities of the time. At this day it would be difficult to find any landholder, or steamboat man, to assert the same things. The railway has traveled on, and more than reached the position then claimed.

EUROPEAN TOUR.—On the closing of my connection with the Hudson River Railway in the spring of 1850, I concluded to make a trip to Europe. My health had been impaired by severe and long-continued labor, and I had for some time indulged a desire to make such a tour, hence I embraced the opportunity. The trip was of great interest, and afforded me much pleasure.

It so happened that while I was there, one of the large tubes of the bridge over the Menai Straits was to be launched. Mr. R. Stephenson kindly gave me an invitation to witness the operation. The spectacle was highly interesting in itself, and was followed by an invitation to dine with a party of English engineers, an occasion I enjoyed very much. I noticed that they far excelled me in making speeches, which was quite the order at dinner. It was an art I had not cultivated, and I expressed to them my surprise at the skill they exhibited in this respect. It is not worth while to go into particulars of this tour. I will simply state that my observations were mostly on engineering work which interested me very much.

I returned after an absence of four months, greatly improved in health, and immediately engaged in the construction of the

MICHIGAN SOUTHERN & NORTHERN INDIANA RAILWAYS.—They were practically one railway, about 246 miles long, extending from the head of Lake Erie to Chicago, on Lake Michigan; about 66 miles had been constructed by the State of Michigan, with a timber rail and iron plate. It was now in the hand of an incorporated company, which had a section of 18 miles nearly completed, and laid with iron rails. I spent most of the winter of

1850-51 on the line of this railway. There remained about 160 miles to be constructed. The route was very favorable. This portion of the work was entered upon in the spring of 1851, with vigor, and in about one year, a line was opened to Chicago. The principal difficulty was one of finance. Though the work was of very moderate cost, eastern capitalists were very reluctant to enter on an enterprise in the western States, as several had repudiated their debts, and this impaired confidence in the honesty of their institutions.

During the summer of 1851, I engaged as President, in the construction of the Chicago & Rock Island Railway, extending (as a continuation of the Northern Indiana Railway) from Chicago to the Mississippi river at Davenport, 180 miles. This was also an easy line to construct, and was brought into use in 1854.

I continued my connection with the Michigan Southern & Northern Indiana Railway until the spring of 1858, when it became obvious my views of management could not prevail. With the exception of incidental services, I had no professional engagement until December, 1861.

PITTSBURGH, FORT WAYNE & CHICAGO RAILWAY.—This road was 468 miles in length, extending from Pittsburgh, to Chicago. It had been constructed under financial embarrassments, and as usual in such cases, was poorly built. There were first and second mortgages on the railway, for about \$10 000 000, and a floating debt of between \$2 000 000 and \$3 000 000; the capital stock was about \$6 000 000—making a total liability of about \$18 000 000. The company defaulted on their interest, and after two or three years, the bondholders foreclosed and sold the railway in the early part of the autumn of 1861. From inferior construction and inadequate maintenance, the line, at the time it was taken in charge by the Trustees of the bondholders, was in very poor condition.

In this state of affairs, I was appointed General Superintendent. The stock and floating debts were very honorably provided for by the Trustees. For the floating debt, a third mortgage was given to the holders of claims, conditioned that no interest should be paid until a sufficient surplus had been earned therefor, after providing interest and sinking fund for the first and second mortgage bonds. After the net income should thus provide as above, for all the bonded debt, then the stockholders should be entitled to any balance obtained from net income. I mention these particulars as very creditable to the Trustees, and in striking contrast with the action of bondholders in some other cases.

The capital stock was very low in the market. I was informed that it had been sold at 8 per cent. Not long after I had entered on the management, a stockholder enquired of me to know if I thought advisable to sell at 20 per cent. I replied that I could not advise in such cases—that I regarded the prospect of traffic as good, but the poor condition would require large expenditures to put the railway in even reasonable order.

The Trustees were extremely solicitous to secure such net earnings as would provide for the interest and sinking funds of the first and second mortgage bonds, saying, that beyond such provision they were willing all earnings should go to the improvement of the railway. In order to secure their object, the Trustees required a monthly sum to be placed in their hands.

I will not detain you with further details, but merely give the financial result. The first year passed with decided expression of satisfaction, and at the end of the second year, there were further net proceeds. After 2½ years, I resigned the superintendency, but continued to act as Engineer until 1866. The bonds, in interest and sinking funds had all been provided for in these 2½ years, and such surplus was in hand, that soon after, the Board of Directors declared a dividend on the stock at the rate of 10 per cent. per annum, which rate has not since been reduced.

ENGINEERS AS RAILWAY SUPERINTENDENTS.—Now, it may be said, why all this about the superintendency of a railway—engineers are not much engaged except in construction? My object is to show that an engineering education fits a man best for the superintendence, no less than for the construction of a railway. I regard it an error that the profession has not given more attention to superintendence of operations. It demands the same thoughtful analyzing mind, to operate, that constructs. I have presented this case, to impress the importance of employing the best engineering experience and ability in the superintendence of railways. The case is a plain one. Say, management has failed to pay interest, and the railway was sold by foreclosure of mortgages. An engineer takes charge, and in 2½ years this is all changed and the property becomes a success. But why an engineer? A true engineer first of all, considers his duties as a trust, and directs his whole energies to discharge the trust, with all the solemnity of a judge on the bench. He is so immersed in his profession that he has no occasion to seek other sources of amusements, and is therefore always at his post. He has no ambition to be rich, and therefore eschews all commissions that blind the eyes and

impair fidelity to his trust. His ambition is to make his trust a success to the proprietors. His education fits him to analyze facts, and more successfully than a layman to produce the best results of administration. To reach the best results of railways, they must be put under the superintendence of competent engineers.

It may be inquired, why should not engineers seek to be rich? certainly, they need the benefit of pecuniary resources as well as others, which is not denied. *Rich* is a relative term. The man who, by a proper economy, is able annually to lay something by as an investment from his earnings, is only exercising the prudence necessary to provide for age or infirmity. This all men should do, and it is especially important for an engineer, that by this means his mind may be at ease in regard to the future. What is meant by a thirst to be rich, is a speculating turn of mind, which is sure to prevent any especial eminence in an engineer. He should be careful to secure a proper salary, and then give his undivided attention to the duty he assumes. I have known several engineers who were always on the alert for speculation, but I have not known one such to secure any valuable standing as an engineer. To this it may be said, engineers have not, in all cases, succeeded in conducting the operating business of railways. The same may be said of lawyers, or any other profession, and it is just as important that the party in interest select a competent and faithful engineer as a competent lawyer. The same discretion is necessary in one case as in the other.

Being now past four score years of age, I can have no other motive than the public good in the above remarks. I make them in view of much personal experience, which has brought me to the conclusion, that the railway and kindred public works will not realize their legitimate benefits until their supervision is committed to competent engineers. In order for this, it is necessary to cultivate the profession so as to secure the qualifications and character before named, to carefully avoid all theories that are not based on well established and thoroughly analyzed facts, and to be very cautious before applying any theory, to see that all the facts involved are obtained, and that none are hastily and inconsiderately assumed. The profession is one of great importance to the public interest, and no pains should be spared to fully qualify those who pursue it for every engineering emergency, and to fit them for the discharge of every duty with scrupulous fidelity.

DISCUSSION ON THE

CROTON WATER WORKS AND SUPPLY FOR THE FUTURE.*

MR. J. JAMES R. CROES.—Referring to discussion on conduits, Mr. Theodore B. Samo, Assistant Engineer, Washington Aqueduct, writes :

"I see that Mr. Hutton† referred to me as authority for the statement that on the Washington Aqueduct where the conduit leaked over an embankment, the leak was stopped by raising and widening the embankment. His statement is correct. It may interest you to know that at the time of the leak the embankment was 14 feet wide, and that I had it widened to 30 feet, and raised, so that there is not less than 5 feet of earth on top of the arch, including the Macadam road—12 feet by 1 foot deep."

"The width of 30 feet was not adopted simply to stop the leak, but for the convenience of travel, as the conduit road has become in summer a great thoroughfare. I will not assert positively that the conduit does not leak, but at all events no water shows through the embankment. The embankment referred to is the first one above the distributing reservoir."

Two high embankments near Cabin John bridge used to leak, and were treated in a similar manner, with like results, the roadway being made 25 feet wide. The flow line of the distributing reservoir is kept at such a level that from Cabin John to the reservoir, the conduit is under a head. The head at the outlet is generally 4 feet, and has been as high as 5.8 feet.

ERRATA.—Volume V. On page 395, thirteenth line, for "where" read "when"; on page 396, seventeenth line, for "that" read "the"; on page 397, twentieth line, for "15" read "1.5"; on page 403, thirtieth line, for "the" read "on the"; on page 406, eighth line, for "during" read "through"; on page 413, sixteenth line, for "the channels" read "channels," and eighteenth line, for "of" read "off"; on page 417, twenty-sixth line, for "Segonia" read "Sequonia;" and on page 425, twenty-first line, for "convinee" read "convince us."

Also on Plate II, the Jetty at *L*, should be marked "Jetty No. 1," and the one opposite, running out from the reef, "Jetty No. 2."

* Continued from Vol. V, page 273. † Vol. V, page 260.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CXXXVI.

A WATER CONDUIT UNDER PRESSURE.*

By JOHN T. FANNING, C. E., Member of the Society.

PRESENTED MARCH 30TH, 1876.

In the recent construction of water works for the City of Manchester, N. H., it became necessary to provide for conveying 100 cubic feet of water per second, or at the rate of 65 000 000 gallons per diem, from the supplying lake, a distance of 600 feet, to the turbines and pumps in the pump house. A possible draft of the lake to 6 feet below full water level, in a season of severe drought, and a possible thickness of 3 feet of ice upon the water when the lake might be low in December, made it advisable that the conduit should have its top not less than 9 feet below full water level in the lake.

The level of water in the wheel-pit and tail-race, at ordinary heights, was 45 feet below high water level in the lake. The fall was to be used for hydraulic power required for driving the pumps. The peculiar conformation of the land on the line of the conduit, made it advisable that the conduit should lie upon a nearly direct incline from the forebay at the lake to the wheel-pit in the pump-house. That the velocity of flow in the conduit should not exceed 4 feet per second, a sectional area was required equal to that of a cylinder of 6 feet clear diameter.

* The within remarks were suggested by reference, in Paper CXX, "Notes and Suggestions on the Croton Water Works and Supply for the Future," by Benjamin S. Church, C. E., to the risks attendant upon conduits when flowing full.

Within the pump-house, the conduit joins an open topped vertical stand-pipe, extending to a few feet above the level of the lake. The upper part of this stand-pipe is of 30 inches diameter; it is intended to relieve the conduit of undue strains from water rams that might be brought upon it by the sudden closing of a turbine gate. To the base of this stand-pipe, where it is 6 feet diameter, are connected two 44-inch quarter-turn turbine casings, and a 30-inch water-pipe for supplying the suction chambers of the pumps.

The general requirements of the conduit then were—a cylinder 600 feet long, of 6 feet internal diameter, with its axis at the upper end under 12 feet head of water, and its axis at the lower end under 38 feet head of water.

Several designs for the construction of this conduit were prepared and studied in detail. Four of them will be briefly referred to herein. Either one of them would have been placed in a trench of 13 feet average depth and covered with an average of 6 feet of earth above its top, substantially as shown in Fig. 4, page 73.

A cast-iron pipe, of 6 feet internal diameter, in sections 6 feet long, was first considered, having a large number of its joints flanged and the remaining joints of the usual bell and spigot form and caulked with lead in the usual manner.

A wrought-iron riveted tube, of boiler plate $\frac{1}{4}$ -inch thick, was next considered, and was proposed to be lined with a single $\frac{1}{4}$ -inch ring of brick to give it the requisite rigidity to carry its earth covering. This iron shell was to receive two good coats of red lead paint before being covered, and the back filling was to be puddled beneath and around it with care.

A locked-brick tube, constructed of bricks specially designed for this conduit, was next considered, and was proposed to be bedded upon concrete and buttressed along its lower side extrados, by split stone rubble masonry laid in cement mortar.

A wooden tube, constructed of hard pine staves and hooped with wrought iron hoops, was next considered.

The estimated gross cost of executing each of these was:

Of cast-iron.....600 lineal feet, at \$36 00.....	\$21 600
Of wrought-iron, " " " " 38 00.....	22 806
Of locked-brick, " " " " 22 50.....	13 500
Of hard pine, " " " " 15 25.....	9 150

The last named was constructed, and will be described in detail.

The design for the wooden penstock contemplated the use of longitudinal staves, cut from pitch-pine plank, 4 inches thick, each stave being $3\frac{1}{2}$ inches wide upon the intrado, and having its side joints dressed to radial lines, the whole forming a continuous cylinder, (Fig. 3, page 73).

The penstock was constructed in accordance with this design, excepting the 12-foot length joining the stand-pipe and passing through the formations of the pump-house. This section was of 6-foot cast-iron pipes.

This system of pine staves was adopted partly because its first cost would be least, and partly because its material could be prepared and the construction completed ready for use sooner than by either other method, and the work of putting it together could be commenced very early in the spring before freezing nights had passed. This method was not considered constructively superior to all the other methods.

The work of fitting the stave joints was done entirely by machinery. To this end, the writer caused a set of plain cutters to be fitted in a plank matching machine, and so adjusted as to cut both edges of each stave to true radial lines at the same time, which ensured the edges being truly parallel. The planks were first passed through a "Daniel's" planer and surfaced to an even thickness, then sawed into 4-inch square strips, and then jointed in the fitted jointing machine, when they were ready for laying in the trench without further dressing.

Wrought-iron hoops were placed upon the penstock of $\frac{1}{2} \times 2\frac{1}{2}$ -inch and $\frac{3}{4} \times 2\frac{1}{2}$ -inch iron. Each hoop was made in two equal sections, with clamping bolts at each joint. The details of the hoop joints and tightening bolts are shown, (Fig. 5, page 73). The hoops were placed at average distances of 18 inches between centres, being nearer together at the lower end.

When laying the penstock staves, concave forms of plank were imbedded in the bottom of the trench at proper intervals, to receive the lower curve or invert, for about one-third the height of the shell. Several lengths of the lowest line of staves were then placed and aligned, and then other lines of staves successively placed on each side, until one-third the length of the shell was in position. A centering rib was then laid upon this lower section, the circuit of the shell completed, and the hoops tightened.

The staves were of unequal length, but averaged about 13 feet. Care was taken that no two joints came near each other in adjoining lines of staves. Thus a continuous cylinder was formed.

Three carpenters, assisted by three laborers, could lay about 60 lineal feet of penstock per day; no machinery or apparatus other than hand tools being required in this part of the work.

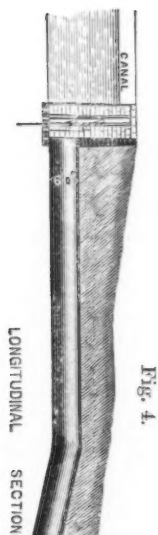
The butt joints in each line of staves, were made water tight by means of a plate of $\frac{1}{2}$ -inch iron, 1 inch wide, let into the centre of each stave end in a saw kerf, $\frac{1}{2}$ -inch deep. These plates, as well as the wrought-iron hoops, were well covered with two heavy coats of red lead paint.

Two slight changes of direction were made in the length of the penstock, to save depth of trench excavations. At the points of change, a continuous joint across the entire shell was necessarily made. This joint was covered with a broad wrought-iron hoop, and caulked with oakum saturated with red lead and oil.

In the original design, it was proposed to cover the entire penstock with a thin layer of fine concrete, to protect it from the oxygen and carbonic acid gas that might reach its upper part through the earth; but the ground on the line of the conduit was naturally moist, even to saturation, and this moisture would be increased by the filling of the long forebay or canal at the head of the penstock, with water. The earth of the trench and backfilling was of such nature that water could but slowly percolate through it; hence it was believed that the thorough saturation of the earth would be an effectual protection to the work from influences of atmospheric gases.

This conduit of wood was completed and put in operation in the spring of 1874, and has now (March, 1876), been in successful use nearly two years, under head pressures of from 12 to 40 feet, supplying the water to the pumping machinery of the City Water Works.

WOODEN PENSTOCK, MANCHESTER WATER WORKS.



VERTICAL SECTION

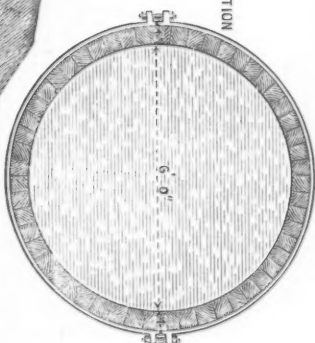
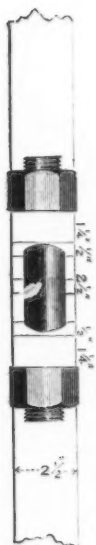


Fig. 5. PENSTOCK HOOP JOINT



CXXXVII.

THE FAILURE OF THE ASHTABULA BRIDGE.

A Paper by CHARLES MACDONALD, C. E., Member of the Society.

READ FEBRUARY 21ST, 1877.

On the evening of December 29th, 1876, at 8 o'clock, an express train, consisting of two engines and eleven cars, going west, broke through an iron bridge on the line of the Lake Shore & Michigan Southern Railway, at the crossing of a small stream, about 500 feet east of the station at Ashtabula, Ohio.

Of the eleven cars in the train, three were loaded with express matter, one was a baggage car, and the remainder were passenger cars of various descriptions, including three sleeping cars. These contained, according to the testimony of the Superintendent of the road, 128 passengers and 17 train hands; of which number, 69 are known to have been taken alive from the wreck, and 72 adults and 8 children are said to have been lost; leaving 4 not accounted for; it is considered doubtful, at present writing, whether these were on the train.

At the moment when the pilot of the forward engine reached the western abutment, the top chord of the south truss, which was almost directly under the train, gave way at a point about 23 feet from the west end, causing the immediate fall of the entire structure; the engineer of the first engine feeling a sudden movement pulled open his throttle valve and succeeded in landing his engine on solid ground west of the abutment, but the remaining engine and the express cars went down with the

bridge, while the passenger cars were dragged one after another over the eastern abutment into a chasm 65 feet in depth, piling one upon the other in a shapeless mass of splintered fragments which immediately caught fire and were consumed.

It is needless to enlarge upon the horrifying details of this occurrence. The community at large have been profoundly agitated by it, and anxious to learn the true cause of the disaster, and the determination of the degree of responsibility, if any, attaching to those who were at the time in control of the road.

The writer visited the scene of the disaster a few days after its occurrence and made a partial examination of the wrecked material, and through the kindness of Mr. John Newell, General Manager, was enabled to make a copy of the original plan and bills of material from which the bridge was built. Again, on February 7th, he made at Collingwood, O., a careful inspection of all the iron work, it having been removed to that point for convenience. Since that time, he has endeavored to prepare the information thus obtained in a suitable form for submission to his fellow engineers, in the hope that in the discussion which may follow this mere statement of facts and some of the inferences to be drawn therefrom, a permanent good shall result not only to the Society, but to the public generally.

For the purpose of systematic consideration, the subject will be presented under the following heads :

1. A description of the bridge ;
2. The circumstances under which failure occurred ;
3. The most important lessons to be learned from the event.

1. *A description of the bridge.*—About eleven years ago, Mr. Amasa Stone, the then President of this part of the Lake Shore Railway, undertook to design a wrought-iron bridge spanning a clear opening of 150 feet, and proportioned to carry the traffic of a double line of rails resting upon the top chords. For some time previous he had been familiar with wooden bridge construction, and was generally considered to be a man of large experience in the requirements of railroad practice, and commanded the confidence of a large circle of influential men in his ability to carry out successfully anything he might undertake to do. The result of his efforts at that time may be described as follows :

A two truss, parallel chord, deck bridge, with the web system arranged as in the well-known Howe truss, but without the end vertical

posts. (Plate IX, Figs. 1, 2, 3.) There were fourteen panels of 11 feet each, with a height of 19 feet 9 inches between centres of chords. The distance between centres of trusses was 16 feet 6 inches, and width of each chord, 34 inches. The first or end panel of braces contained six rolled beams of I section 6 inches deep, 4 inch flanges and $\frac{5}{8}$ inch webs, with a total sectional area of 57.6 square inches. From the first bottom angle block to the centre of the first set of braces were three gas pipe struts $3\frac{1}{2}$ inches in diameter and $\frac{1}{2}$ inch thick, abutting against faced bearings at either end. These pipes contained 2 inch round rods, which were screwed into the bottom castings and passed between the braces to a bearing on cast washers held upon the upper side by a wrought gib plate. The second set of braces were the same in number as the first, and probably of the same section; they were spread far enough apart at the centre line of the truss to allow of the passage of a light 6 inch beam in the direction of the counter brace, and at the intersection they were clamped by two $\frac{5}{8}$ -inch bolts and 3-inch plates. (Plate IX, Fig. 1.) The third set probably had five beams of same general dimensions and one counter; three mains on one side and two on the other, of the counter. The fourth set had four mains and one counter; the fifth set three mains and two counters, and the sixth and seventh sets, the same as the fifth. It was impossible to determine the exact sectional area of the braces *as they were placed in the bridge*. According to the bills of material it was intended to give each beam used as a counter and for centre mains a sectional area of 8.85 square inches, increasing gradually as the strain required, to 11.1 square inches for the end mains, but there appears to have been some modification of the original order, and as all these braces were of the same length, and the erection was intrusted to inexperienced men, it is probable that some of the lighter braces may have been placed nearer the abutments than was originally intended.

The first set of vertical rods contained eight of $2\frac{1}{8}$ inches diameter; the second, eight rods of 2 inches; the third, eight of $1\frac{1}{2}$ inches; the fourth, eight of $1\frac{1}{4}$ inches; the fifth, eight of $1\frac{1}{2}$ inches; the sixth, eight of $1\frac{1}{4}$ inches, and the centre set, eight of $1\frac{1}{2}$ inches. All these rods had solid forged heads at the upper ends and enlarged ends with screw thread and nut at the bottom. They rested upon gib plates 5 x 1 inches, which extended across the chords above and below. The top chords consisted of five I beams 6 inches deep, spaced $2\frac{1}{4}$ inches apart; these rested upon cast-iron angle blocks at panel points, and were arranged so

as to break joints alternately, three on the first block and two on the next. (Plate X, Fig. 7.) From the masonry out to the second block, there were two beams which did not assist in sustaining chord compressions, they were intended to carry the cross floor beams for the extreme panel. The sectional area of each of the end beams was 10.35 square inches, from which there was a gradual increase to 11.85 square inches at the centre, (Plate X, Fig. 8.) They were held together by two $\frac{3}{4}$ inch bolts in each panel, passing through cast-iron ferules, (Fig. 9.) At the angle blocks, those beams which did not break joints were held in place by friction of the gib plate, and at the joints, there were projections *AA*, on the block (Fig. 5), which served the purpose of transmitting web strains. The arrangement of the different sizes of beams, (Fig. 8,) was concluded from an examination of the original bills of material, and from a measurement of some of the beams in the north truss soon after the accident; it is just possible, therefore, that in the bridge as built, this arrangement was not accurately adhered to.

In the bottom chords, there were five lines of flat bars $5 \times 1\frac{1}{2}$ inches each; the two outer lines were in pairs, one above the other throughout the entire span; the next two were single from the ends to the fourth panel, and then double; the centre line was single from the ends to the third panel, thence double. Splices were made as shown in Fig. 6. At panel points, there were lugs, 3×1 inches forged upon the upper bars, fitting recesses in the angle blocks, by which means the web strains were transmitted to the upper half of the chord, and it was assumed that the corresponding lugs on the lower half at the splices would equalize the distribution. The workmanship at splices and connections with angle blocks was very good of its kind, and could not have failed to insure safety at these points. The bottom system of lateral bracing consisted of $2\frac{1}{2} \times \frac{1}{2}$ inch diagonals extending over two panels and connecting with the angle blocks by lugs, (Fig. 2); also struts in the form of railroad bars attached to the gib plates by two $\frac{3}{4}$ inch bolts; these struts did not connect with the chords at same panels with the diagonals except at the first panel, where there was an extra strut. This system had no means of adjustment. Top lateral rods were flat at the ends and connecting with angle blocks, as in the lower system; but they were round in the middle and made adjustable by means of turn buckles. Cross floor beams, of which there were three in each panel, supplied the place of struts; they were connected with the chords by means of yokes

and lugs, (Figs. 9, 10.) There was also a system of sway rods, $1\frac{1}{4}$ inch round with turn-buckles, passing from the top chords diagonally to the bottom, (Fig. 3,) and attached to the same alternate angle blocks as the laterals; this connection consisted of a lug held in a recess in the casting by a $\frac{3}{4}$ inch tap bolt, (Fig. 9.)

The floor of the bridge, (Figs. 2, 3,) consisted of 6-inch cross beams, 9.6 square inches area, upon which rested 7×14 inch-white pine stringers, and 3×5 inch oak ties, spaced 2 inches apart. At the extreme end of the cross beams were 8×10 inch pine guard timbers bolted to them. On the inner side of each track rail was a guard rail of same size, leaving a clearance of about 5 inches; these two guard rails came together at points about 150 feet from the abutments, forming a V shaped guide. Accidents from derailment on such a floor as this are extremely rare.

So far as could be judged of the material without making actual tests, it was all of excellent quality, and no pains seem to have been spared in the forged and machine work to insure accurate connections.

It is said that much trouble was experienced in the erection, by reason of the want of proper supervision, and it is certain that the original plans were modified in some important particulars; all the brace beams were originally intended to be placed with their long side at right angles to the chords of the truss, and there were but four in the end panels, whereas, in the bridge as it was built, there were six beams in the end panels, and all were placed so that their long sides stood upright. The reason given for this change was, that under the weight of the bridge alone, the beams as originally placed buckled and tore apart at the intersection with the counters. This change in the number and position of braces made it necessary to chip off portions of the flanges to prevent them from interfering with the vertical rods, and as the castings had been planed in grooves to suit the first positions, they, too, had to be chipped to as nearly a square bearing as possible. All this work was imperfectly done, and the result was, that the braces did not have what would be understood as square bearings. In the top chords, $\frac{1}{4}$ inch shim plates were forced between the ends of the beams and the cast lugs, but as this was probably due to an error in length of the chord, and pieces were inserted to secure proper camber, they did not necessarily impair the connections. These cast lugs, however, had been reduced in thickness from 2 to $1\frac{1}{8}$ inches, for some unknown reason, and their strength to transmit the web strains was in consequence materially lessened.

In Fig. 8, will be found a diagram with two sets of strains for one truss marked upon its left half, and the sizes of material on the right; the largest strains are based on an assumed rolling load of 2 000 pounds per foot on each track, and a dead load of 1 415 pounds per foot of single truss, the web strains being modified by taking proper account of the excess of engine load per panel. The numerals *underscored* represent the actual strains produced in the south truss by the train which caused the accident; they are given for comparison merely, and to show that although this was a double track bridge, almost every train passing over it affected either one truss or the other nearly up to a maximum. The numbers with "□" attached are the strains per square inch upon the material in each panel produced by the assumed rolling load of 2 000 pounds. The dead weight given, was determined from the weigh-master's returns of iron taken out of the wreck, and by assuming that the oak ties weighed 4 pounds per foot board measure, and the stringers 3 pounds.

A discussion of this diagram will appear more appropriately under the second division of our subject, which we will now proceed to consider.

2°. *The circumstances under which failure occurred.*—In a blinding snow storm, with the wind blowing a gale from the north, the ill fated train under a speed of 15 miles an hour moved forward upon the bridge; at a moment of time which is not distinctly impressed on the mind of the engineer of the first engine, he felt the structure give way under him and as it were by instinct, he threw open the throttle valve and succeeded in reaching the west abutment with the wheels of the engine on the track while the tender was derailed; looking back he saw the bridge and train piled up at the bottom of the ravine in indescribable confusion. The alarm was at once given, and every possible effort made to extend aid to the unfortunate passengers, but the prevalence of the storm and the fact that all the cars contained stoves filled with burning coals, rendered this task an almost hopeless one, and in an incredibly short space of time all that was combustible had passed out of sight. The second engine was found near the southwest corner of the abutment, lying on its side and pointing nearly east, with the pilot uninjured and the draw bar, by which it had been connected with the forward tender broken square off. The southwest corner of the abutment showed undoubted signs of having been struck by a heavy falling body, and from the position of the wreck of the second engine with reference to it, it is

evident that that engine must have moved forward on the track until the forward truck wheels were directly over the abutment. It was held in that position for an instant, until the rear, swinging down with the bridge, struck the abutment, thus causing the snapping of the draw bar, which was then under a severe cross strain, and as the pilot end was liberated the whole engine capsized towards the east. The south truss, with the exception of the second panel from the west, fell to the north with part of the top chord under the bottom chord of the north truss; the second panel of the top chord fell to the south and was covered by the engine. The north truss fell to the north, and the whole bridge or perhaps more particularly the south truss, moved about 8 feet eastward; all the cars except the last sleeper landed about the middle of the ravine and to the south; the last sleeper before reaching the east abutment rolled over to the north, crossing the down track and plunging down the embankment was consumed at a point about 150 feet northeast of the others. Very little of the iron was broken, and a much larger percentage of beams were taken out comparatively straight than might have been expected. The two beams extending outward from the masonry, (Fig. 2) were straight so far as the first angle block, then bent at a sharp angle 45° to the south, then straight to the end; the three beams reaching from top of the first set of braces to the third set were bent south around a curve of 90° . The east iron angle block at top of second set of braces had the south lug broken off close to the face, and the line of fracture disclosed an air hole extending over one half the entire section. It is said that this casting was found resting upon the bottom chord having slipped the whole length of the vertical rods. The second set of braces were all bent to the south, and were nearly doubled upon themselves. Several other chord castings were broken into small fragments, and some of those which were not broken in the body, had one or more lugs knocked off.

From the facts stated, there would seem to be little doubt but that failure first began in the south truss, at the second panel point from the west abutment. This panel must have moved to the south, while the first and third points were held in position for the moment, by the vertical sway braces and laterals, until sufficient distortion had taken place in the chord, to cause complete failure of the bridge. That the end braces did not move first, is shown by the fact that the first engine pulled clear, and also by the distortion of the end chord beams at the point where they passed the first casting; these show that they were held as in a vice till their free ends were bent 45° .

The second set of braces could not have buckled at the first instant, as this would have immediately depressed the second panel sufficient to bend the end beams downwards as well as outwards, the same result would also have been noticed in the three chord beams reaching from the top of the first set of braces; in neither case, however, is there any evidence of bending except in a horizontal direction. If the failure began by buckling of the top chord between first and third set of braces, it would probably be indicated by an excessive strain per square inch upon the material, but by referring to the strain sheet it will be seen that this was very nearly the strongest part of the top chord. It would seem, therefore, to be the only alternative, to assume that destruction began in the second top chord angle block. By referring to Fig. 5, it will appear that the horizontal component of the strain from the second set of braces must necessarily pass to the only two beams in the chord, which are in a position to receive it, through the cast-iron projections *A A*, near the centre of the block; these were each 6 inches wide by $1\frac{1}{8}$ inches thick, and, as we have already noticed, the one which was found broken off was so far impaired by an air hole as to be reduced in strength fully one-half. The amount of strain which this lug had to transmit must have been 53 000 pounds at the time of the accident; and considering the manner in which it is applied and the fault in the casting, this spot would appear to have been the weakest in the bridge. At no other point were these lugs subjected to so great a strain, except at the end casting and here they were heavily reinforced. There is one fact, however, which must not be lost sight of in this connection: the broken lug was on the south side of the block, and at first thought this might indicate that the point should have gone to the north, if failure began in the manner suggested.

An examination of the strain sheet reveals the fact that the bottom chords and vertical rods were ample in strength, and it is not believed that the material or workmanship was in the least defective; but in passing to the compression members a serious difficulty is encountered in the attempt to determine a factor of safety from the strain per square inch, owing to the uncertain character of the data to be used in Gordon's formula, for the strength of columns. Let it be assumed that for columns having square end bearings, this expression would be for American iron;

$$\text{Breaking weight per square inch} = \frac{40\,000}{\text{length}^2};$$

$$1 + \frac{40\,000 \times \text{rad. of gyration}^2}{\text{length}^2}$$

and for round ends, one-third of this amount. The value of radius of gyration, squared for a beam of 9.6 inches section, is 0.835. Substituting this in the equation, and taking length for braces 260 inches, (inasmuch as the connection at centre is very imperfect,) we should have 13 300 pounds per square inch as the *breaking resistance* of one 6-inch beam having square end bearings and a length of 21 feet 8 inches. If it be assumed, that owing to imperfect end bearings, these braces had virtually round ends, the breaking resistance, 4 433 pounds, would be below all but the two centre sets in the bridge. The structure never would have even carried its own weight for a single day. But the facts disprove this assumption, and, unfortunately, we have nothing to guide us in deciding upon any figure between these limits.

Again, in the top chord it is impossible to say what effect the attachment of the beams to each other and of the floor connection, had in increasing the resistance over what the unsupported beams would give by the formula. It is certain that the top chords in the centre panels must have been strained up to 8 700 pounds per square inch, thousands of times without giving any perceptible signs of weakness, while failure finally took place in the strongest part of the chord, probably from a defective detail. It would appear, therefore, that in these members, the strains were in excess of good practice, and they had never shown any evidence of weakness. Of the braces, it will not be possible to speak so confidently. There can be little doubt but that the end panel stood firm until the destruction of the next panel. These braces were strengthened, however, by the gas-pipe struts to a much greater extent than any of the others, and of these, all that we shall ever probably know, is that the strain per square inch on the beams where failure began, was less than at any other set except near the middle.

3°. *The most important lessons to be learned from the event.*—In the interval since the accident, we have had a sufficiency of snap judgments to satisfy the most censorious. Judging from the tenor of much that has appeared in the secular press, either as evidence taken under the solemnity of an oath or by way of editorial comment, this bridge must have been conceived in sin and born in iniquity.

The President of the Company attempts to execute a difficult piece of construction, with but little special knowledge of the principles involved in his task. He ignores the advice of a chosen professional assistant, and neglects to profit by the warnings which are said to have been uttered by

the structure itself in the travail of its birth, and now, at the end of all these years, a dire catastrophe brings the misshapen thing back to the source from whence it sprung. A dark enough picture this, but let us look into it more closely. Twelve years ago, what was the *extent* of knowledge possessed by engineers on the subject of wrought-iron bridge building, judged by the work done, rather than what might have been derived from books? On the New England roads there were practically no iron bridges; that great trunk line, the Boston & Albany, still revelled in the security afforded by "the principle of the Howe truss," in wood; the New York Central, under the guidance of a foreign engineer, was experimenting in riveted work, now so much written against and used; none of the roads centering in New York had substituted iron for wood. The Pennsylvania Railroad, almost alone in that State, was but in the infancy of the effort which has since resulted in securing to her use some of the finest specimens of bridge architecture in the world. In the West a few scattering efforts had been made, and the subject was beginning to attract the attention of some of the best minds in the country. Squire Whipple, Albert Fink, Shaler Smith, Jacob H. Linville, and Thomas C. Clarke had built bridges at that time, it is true, but such names could almost be counted upon the fingers; and even these would, perhaps, now admit that they then "built better than they knew." If then, the state of knowledge at the time has not been under-estimated, the Ashtabula bridge was the result of an honest effort to improve the bridge practice of the country, undertaken by a man whose experience in wooden bridges warranted him in making the attempt. As to his willful neglect of proffered advice, it would be well to suspend judgment until all the facts are brought to light by the proper tribunals. His worst enemies will at least accord to Mr. Stone, the possession of common sense.

After successfully overcoming the many difficulties incident to all new undertakings, the bridge was completed and tested, with results which were supposed to be satisfactory. It passed into the hands of the regular inspection officers of the road, who, from current report, were most careful men, but who had never had more than the ordinary practical experience gained in the handling of wooden bridges. In due time the bridge failed, and we must know the reason why.

First. The inspection must have been faulty. If any one of the well known bridge engineers of to-day had been asked to examine that structure he would have pronounced it unsafe, for the principal reason that

all the compression members were liable to fail by flexure. Why, then, was not such inspection resorted to in time? The answer is, *that the management of not one road in fifty was aware of the necessity of incurring such expense.* The Pennsylvania Rail Road stood almost alone in the practice of employing scientific experts as inspectors. The New York Central and the Boston & Albany profited by the fact that the original designers of their riveted work still remained to watch the results of their experiments; but these were the exception. On almost every important line precisely the same system was in vogue as that which has just been described.

The most important lesson would then appear to be, that structures involving the application of scientific principles should be intrusted to scientific experts in their construction and subsequent inspection, and managers of public works must now learn that cheap men are not necessarily the most economical. Just here it may be proper to say one word "*in reductione ad absurdum*," to the suggestion that inspectors should be chosen from the ranks of the engineers of the army and clothed with governmental authority. The same argument which is advanced in respect to bridges would apply with equal force to every form of construction included under the name of public works. We should not only have government inspectors of bridges, but also of rails, ties, road-bed, engine houses, depots, and, possibly, even of boards of direction: and then indeed, would we begin to realize the vast scope and magnitude of Hans Breitman's great moral idea, which was, "dat government for every man moost alfays do every dings."

Second. A careful study of the behavior of the compression members of this bridge must impress us with the necessity of more perfect experimental knowledge of the strength of iron in the form of struts. Almost the only information we now have, is crystallized in that somewhat flexible expression known as Gordon's formula, and we have seen how difficult it has been to extract any comfort from it on this occasion. The thanks of the profession are due to President Grant for his special efforts to induce Congress to strengthen the hands of the United States Commission to test Iron and Steel.

Third. The failure of some of the castings conveys a useful lesson in designing details involving the use of cast-iron. Care should always be taken not to pass abruptly from a large to small mass, else the strains from cooling will surely vitiate the strength of the connection. The

air-holes found under almost every break are but the result of unequal cooling in the main body of the block and the lug.

Fourth. In conclusion, it may with safety be said that the Ashtabula bridge was an exceptional structure, both in its design and execution, and that the reputation of American engineers and bridge constructors of to-day, cannot in the least be affected by its failure when all the facts are known. Neither should it be assumed that wrought iron, as a material for construction, has shown itself by this experience to be questionable; no evidence whatever can be found in the broken fragments at Collingwood, of any internal or structural change in the fibre by reason of the excessive strain to which the iron had been so frequently subjected. But the fact exists, that the inherent weakness of this bridge had escaped the notice of a regular inspection which was generally believed to be adequate, and the result undoubtedly points to the immediate necessity of a radical change in a system which has shown itself to be utterly unequal to the requirements of the age. What the nature of that change shall be, is a subject demanding the most careful consideration.

From a somewhat extended acquaintance with bridge construction, the writer is of the opinion, that managers of railroads should take the initiative by at once instituting a thorough inspection of all their structures, employing for the purpose, engineering experts whose special experience has been in the design and construction of roofs and bridges; and further, that such inspection should be made regularly at least twice in each year.

MR. EDWARD S. PHILBRICK.—The great mystery is, that this bridge held together so long. The top chord members, 20 to 30 feet from the west end, appear to have been subject to a compressive strain of nearly 6 000 pounds per square inch by the actual load on the bridge when it failed, while they had no lateral support worth naming for 22 feet in length, and must be considered as five separate I beams, acting as struts, 22 feet long, with no probability of equal duty among the five. Moreover, the deflection of the floor beams under an engine would bring their load on the one or two inside members of the top chord with a transverse strain rather greater than prudent, even if this were their only duty in addition to the above named longitudinal stress. The frail attachment of the lateral bracing at this last western panel,—which, owing to a special splice in the $2\frac{1}{2} \times \frac{1}{2}$ inch straps all depended on a single $\frac{3}{8}$ inch bolt,—may have parted from stress brought on the system from wind on the train, leaving

the top chord unsupported laterally, for an indefinite length, except by friction of floor. The angle blocks show that some of the braces had a very firm bearing, as the blocks were polished by the pressure, while their neighbors showed paint that had been run in between block and brace, and had not been worn off, indicating that some braces bore much more than their share of the load.

The sketch shown in Plate XI, is taken from the note-book of my assistant, Mr. Howland, who made a personal examination of the wreck under my direction. It indicates quite clearly that failure must have taken place at the second panel from west end, as Mr. Macdonald says. Attention is particularly called to the set of braces doubled up at this second panel, and if a line is drawn between top ends of vertical rods it will show how top chords would have laid if they had not been scattered by the fall. I hope the disaster may serve to teach railroad managers not to attempt too much engineering themselves.

MR. THOMAS C. CLARKE.—The thanks of this society are due to Mr. Macdonald for taking pains to verify the facts relating to the mode of construction and the fall of the Ashtabula bridge, and for the concise and clear manner in which he has presented them. Such papers are of the highest value, and the more had the better.

From what he has said, the conclusion seems inevitable that the bridge failed at the second angle-block from the west end, by the braces breaking, probably in a lateral direction. It appears highly probable that his suggestion that the breaking of the lugs on the top of angle block was the inciting cause of the yielding of the braces.

From Mr. Macdonald's strain sheet we find that the dead load caused a strain of 49 tons on the original four braces in the second panel, or 12.25 tons on each. We are told that after erection when the wedges were struck away, the clamps confining the main braces to the counters parted, *allowing the buckles to buckle and the truss to sag*. The braces were then turned half round, and two additional ones put in; the bridge then stood and carried its traffic for ten to twelve years. When it broke, we find from Mr. Macdonald's diagram of strains, that these six braces were carrying a dead and live load of 208.8 tons, equal to 18.13 tons each. In other words, when not clamped to the counters, these were columns of 96 diameters long, and broke with 12.25 tons. After being clamped securely, they were columns of 48 diameters, and loaded with 18.13 tons each.

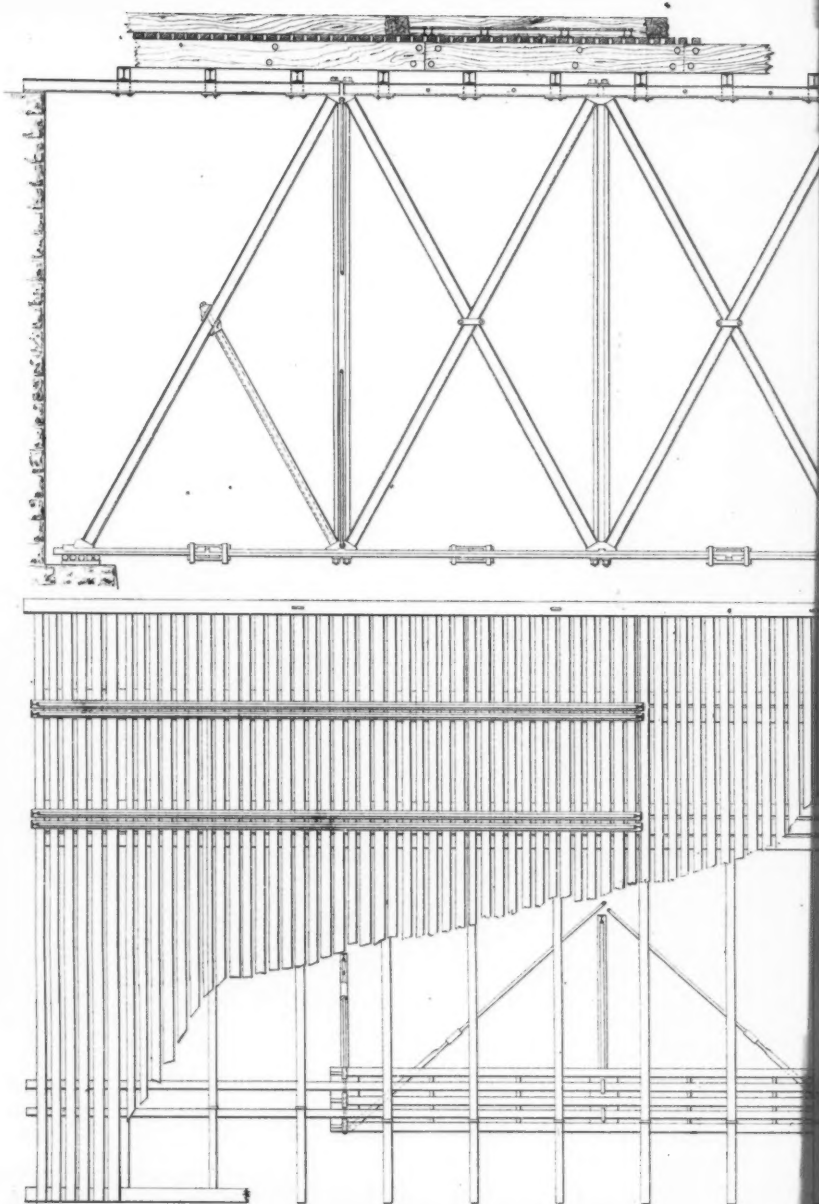
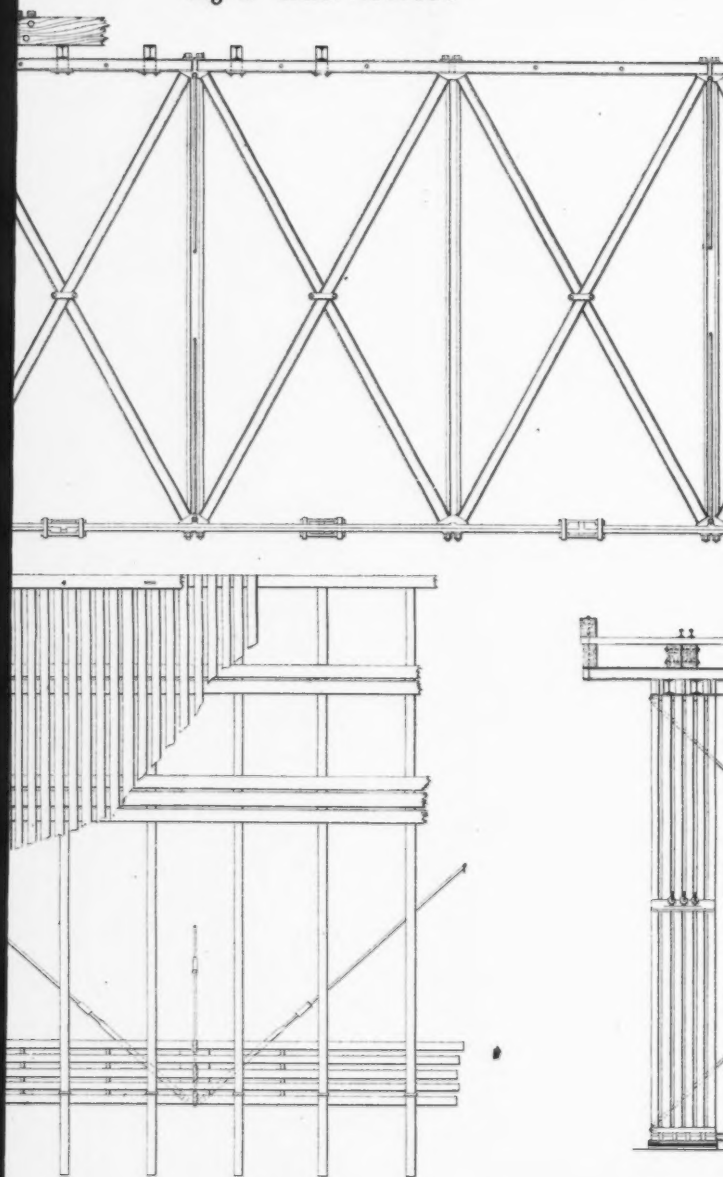


Fig. 2. PLAN.

Fig. 1. ELEVATION.



12 0 2 4 6 8 10 12 Ft.

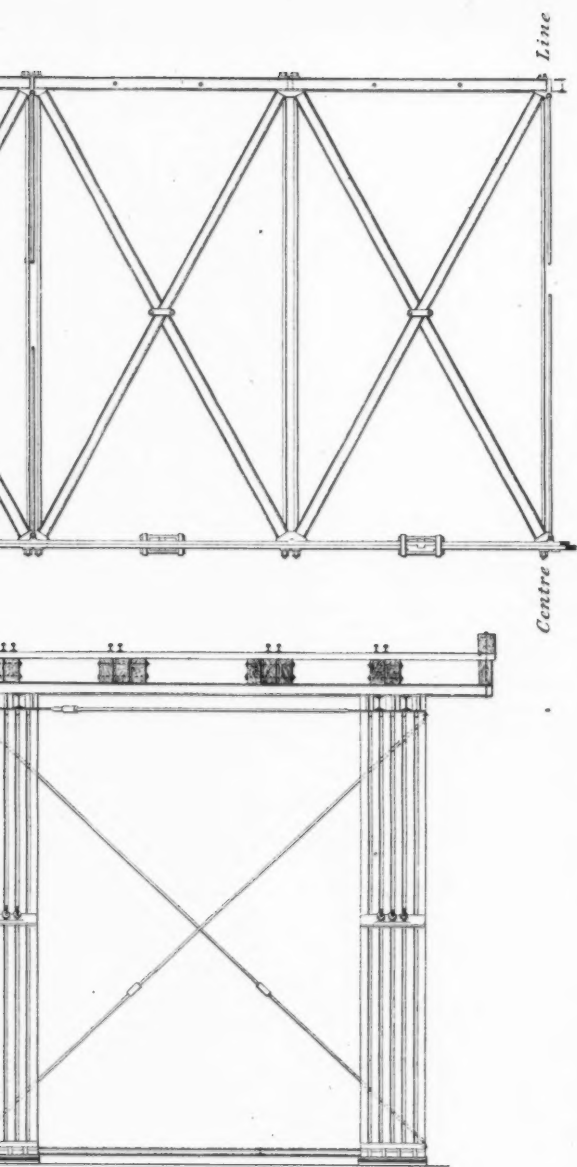
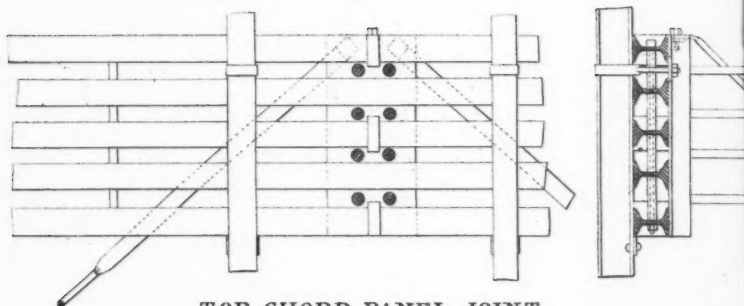


Fig. 3. END VIEW.

Fig. 10.



TOP CHORD PANEL JOINT.

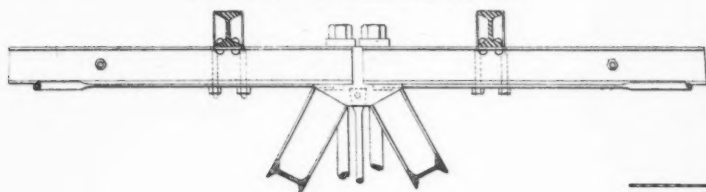


Fig. 4.

Fig. 7. TOP CHORD.

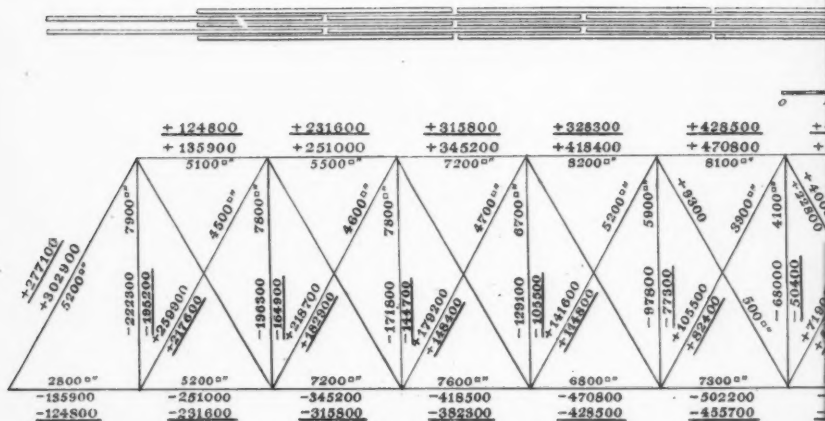


Fig. 8. DIA

Fig. 9.

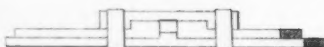
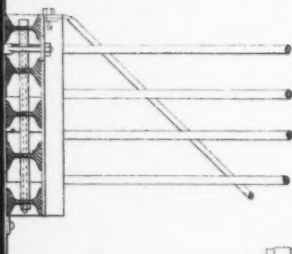
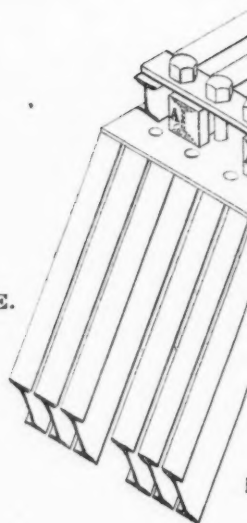


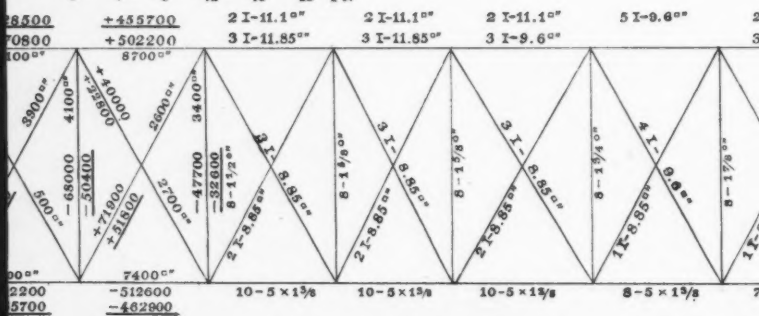
Fig. 6. BOTTOM CHORD SPLICE.



12 0 1 2 3 4 5 Ft.



0 4 8 12 16 20 Ft.



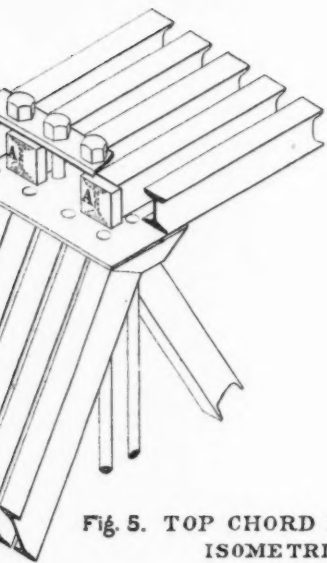
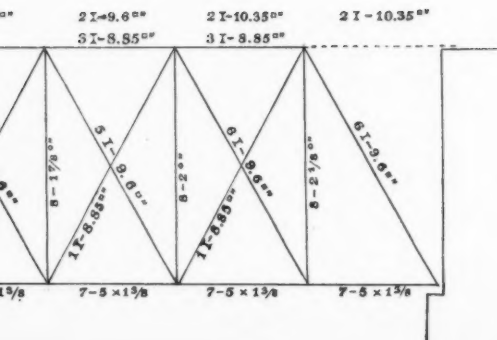
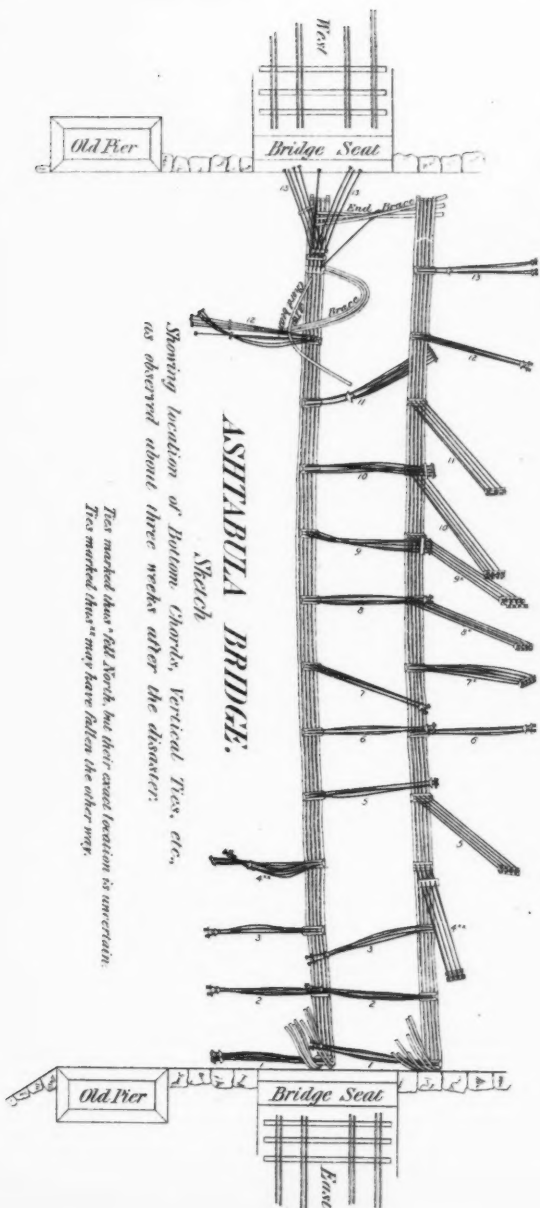


Fig. 5. TOP CHORD PANEL JOINT.
ISOMETRIC VIEW.







ASTABULA BRIDGE.

Sketch

Showing location of Bottom Chords, Vertical Ties, etc., as observed about three weeks after the disaster:

Ties marked thus "Old North, but their exact location is uncertain. Ties marked thus "may have fallen the other way.



According to Gordon's formula, the 96-diameter columns, *if square ended*, should have sustained 25 tons, but these yielded with half as much. When their length was reduced one-half, their ultimate strength, taken as before—equal to one-half of that of a square-ended column—should have been 34 tons, but they yielded under about one-half of that, or 18.13 tons. This indicates two things; *first*, that their bearings on the angle blocks were more imperfect than originally; we are told that to gain room to place six braces where four were originally intended, the flanges were chipped away from several of them; *second*, that as Mr. Macdonald has told us, the angle blocks slipped or shifted out of position.

The conclusion of the whole matter seems to be that, the problem presented to us is, not to account for the bridge falling, but to explain why it stood up so long as it did, its points of weakness were so many. These weak points were so apparent that any bridge expert would have condemned the structure almost on sight. Most of us would have done so, *a priori*, without inspection, simply on the ground that it was in a state of unstable equilibrium, its parts being held together by the friction of the ends of the braces on the angle blocks, and not by rivets or pins.

Finally, the fact that these defects did escape the vigilance of the officers of the road, whom we know to be faithful and competent, seems to point to the necessity of outside inspection by experts. This inspection, to be of use, should be general, covering all bridges; should be made according to a uniform system, and finally, by men in whose decisions both the railways and the public would have confidence.

CXXXVIII.

CO-ORDINATE SURVEYING.

A Paper by HENRY F. WALLING, C. E., Member of the Society.

READ FEBRUARY 7TH, 1877.

RELATION OF SURVEYING TO GEODESY.—Surveying is commonly defined as the art of measuring, laying out and dividing land. The similar but more comprehensive art of geodesy applies to the earth itself, as indicated by the etymology of the word, which is derived from *γῆ, the earth* and *δαίω, I divide*. Its objects are, the determination of the form and dimensions of the earth and of its different portions; the continents and islands with their lakes, rivers, mountains, civil divisions, etc. Its processes, like those of astronomy, upon which, indeed, it is to a considerable extent dependent, require instruments of a high degree of precision and mathematical computations of great refinement.

GENERAL GEODETIC CO-ORDINATE SYSTEM.—In the operations of the great geodetic surveys of the world, including for our own country those of the United States Coast Survey, positions are finally determined by referring them to co-ordinate planes. The plane of the earth's equator forms one of these co-ordinate planes and that of an assumed standard meridian another, the origin being the centre of the earth or the point where the earth's axis cuts the equatorial plane. The position of any geodetical point is fixed by determining the direction in space of its radius vector, as referred to the two co-ordinate planes, and the length of this radius vector. Its angle with the equatorial plane measured on either side is the latitude; the angle of its projection

upon the equatorial plane with the meridional plane, measured in either direction around a semicircle is the longitude, and the length of the radius vector, or rather its excess over that at the level of the sea is the altitude, of the place determined.

VARIATION BETWEEN OBSERVED AND TRUE LATITUDES.—Owing however, to the spheroidal form of the earth, latitudes as observed and recorded do not exactly represent the co-ordinate angles or true latitudes. The observed latitude of a place is the angle made by a normal to the earth's surface at that point with the equatorial plane. The co-ordinate angle could easily be computed from the observed latitude if the earth were an ellipsoid of revolution of known eccentricity.

IRREGULAR FORM OF THE EARTH.—But it has been found that this is not the true form of the earth. The equator, and probably all parallels of latitude, taken at the level of the sea, vary more or less from true circles, indicating a want of homogeneity in the materials which make up the earth's volume. Longitudes are measured by noting the earth's rotation angles on the undoubted assumption that its angular velocity is strictly uniform. By noting the difference in time between the passage of a normal to the earth's surface at any particular point, and of another normal at any point on the standard meridian, across a celestial meridian, we obtain an angle which is called the longitude of the former point, the longitude of the standard meridian being zero. Irregularities in the form of the earth of course change the direction of normals to its surface, and require, both for latitude and longitude determinations, corresponding corrections.

ITS DETERMINATION A DIFFICULT PROBLEM.—In consequence of these irregularities, the problem of determining the exact form of the earth is an exceedingly difficult one. Instruments of the highest degree of precision must be used by skilled observers, and the combined results of a vast number of careful observations over widely extended areas must be subjected to the most refined mathematical investigation before its full solution can even be approximately obtained.

PROGRESS MADE IN ITS SOLUTION.—Its investigation has been going on however for one or two centuries, and very fair progress has been made. The great national surveys of the world have been conducted by men eminently qualified for the task, and the results of these surveys so far as completed, seem to approach quite near to the attainable limits of accuracy.

UNITED STATES COAST SURVEY.--This is particularly the case with our own Coast Survey, which is probably unsurpassed, if not unequalled in precision and general accuracy. Our country, however, has thus far failed to realize some very important advantages which might be derived from these Coast Survey operations, although they have been sufficiently carried out to render them available over considerable portions of the country.

OBJECT OF THIS ESSAY.—It is the object of this paper to point out a simple method by which the high degree of precision which accompanies the Coast Survey work may be made available in the ordinary operations of land surveyors and civil engineers in those districts over which the Coast Survey triangulations have been carried, and at the same time to call attention to the importance of an extension of these triangulations over the entire country, either by the general or by State governments. One of the disadvantages of our peculiar confederate form of popular government is an apparent inability or indisposition to undertake works of acknowledged and eminent utility, unless they are popular with the masses or with those who control the machinery of elections. It is certainly the experience of foreign countries, including several less wealthy and prosperous than our own, that these surveys are many times more valuable than their cost, in the aid they afford to the carrying out of internal improvements, to the equalization of taxes, and to many of the necessary operations of general and municipal governments, as well as of private individuals. But in this country where legislation usually follows, instead of leading, the expression of general public opinion, such works are likely to await the slow and gradual cultivation of the popular mind to a proper appreciation of their great utility.

CONGRESSIONAL LEGISLATION.—In the meantime, it must be admitted that Congress has enacted a very liberal and wise law by which the Coast Survey is authorized to extend its triangulation over any State in which scientific surveys have been provided for by the State Legislature. Moreover the Superintendent of the Coast Survey evinces a disposition to construe this act with great liberality. It is understood that he will, where thorough topographical surveys are undertaken by State authority, cause the subsidiary triangulation to be carried to any reasonable degree of minuteness with the same refined accuracy which has characterized the work already done by the officers of the survey. Indeed, with their elaborate determination of local irregularities in the form of the earth,

and their well organized corps of skilled observers and computers, they have an immense advantage, which could hardly be soon obtained by any new organization, for extending the triangulation over whole States and carrying it to the secondary and tertiary stages.

TRIANGULATION OF MASSACHUSETTS.—The State of Massachusetts is entitled to the credit of being the first of the United States to recognize the importance of having her territory carefully surveyed and to commence upon such a work. More than forty years ago, a triangulation was commenced which was completed in 1842. This triangulation will compare favorably with the Coast Survey work and with the other geodetic surveys of the world. But a surprising indifference to its value, or potential utility, seems to have prevailed, and up to the present time, no further use has been made of it than to adopt it for the basis of such State and county maps as have from time to time been published. These maps give only the horizontal locations of objects obtained from such imperfect surveys as could be paid for by the sale of maps, published entirely by private enterprise, no assistance being given by the State.

STATE SCIENTIFIC SURVEYS.—At the present time, the subject of scientific surveys is being agitated in the States of Massachusetts, Rhode Island, Connecticut and New York. The latter State appropriated \$20 000, in 1876, for preliminary organization, to effect which it appointed a skilled director. The other States mentioned have organized commissions to investigate and report on the subject.

REFORM NEEDED IN LAND SURVEYING.—Before proceeding to describe the proposed combination of surveying with geodesy, a brief consideration may be permitted of the urgent need of reform in the prevalent methods of land surveying and of writing descriptions in conveyances of land, based, as many of these descriptions necessarily are, upon imperfect surveys, and frequently upon no surveys at all. Lawyers and land surveyors are perhaps most familiar with the short-comings of these conveyances in failing properly to describe the property conveyed. A large share of the litigation of the country arises from this cause. Indeed the laxity in this respect is something almost incredible. Scarcely one deed of conveyance in a hundred, will be found to contain such a description of the land conveyed as would fix its location with certainty, if the fences, walls or other inclosures should become obliterated, a contingency which is quite likely to arise.

CONFLAGRATION OF DETROIT.—An occurrence of this kind on a rather extensive scale took place in 1805, when the city of Detroit was devastated by fire, and so thoroughly destroyed, that it was found quite impossible to ascertain the former boundaries of estates. Their restoration was entrusted to the Governor of the State and a council of judges, who could find no better way out of the difficulty than by re-laying out the city on an entirely new plan, dividing the lots among the former owners as equitably as possible.

Notwithstanding this experience, the lines of streets and lots in Detroit are now so uncertain that disputes and litigation in regard to them are of continual occurrence. The same is true in most of the cities and large towns of the United States, especially in suburban districts and growing villages where land is rapidly increasing in value.

FAULTY DESCRIPTIONS IN LAND CONVEYANCES.—If we examine the descriptions given in land conveyances, we shall find that they usually fail to fix either the location of the property by references to permanent land marks, or even the position of its boundary lines relative to each other. Frequently the tract conveyed will be bounded in the deed by the several tracts adjoining, of which the only description given is to state the names of the supposed owners. In many deeds, all dimensions are omitted and only an indefinite approximation to the quantity of land conveyed is given, the statement being that it contains about so many acres, "be the same more or less." Where good permanent division fences, walls, hedges, ditches, streams, shore lines, bound-stones, stakes, &c., mark the boundaries, and are properly described, such descriptions may answer the purpose so long as the boundaries remain unchanged, although such indefiniteness as to quantity would hardly be tolerated in the sale of other kinds of property.

OBLITERATION OF MONUMENTS.—But physical monuments are continually becoming obliterated even when well defined at first. It is said to be an old custom in some parts of the country, to take children once in every year to important boundary corners, where monuments have been erected, to the location and surroundings of which the careful attention of the children is directed. If on a subsequent visit, their memory is found to be at fault, it is refreshed and deepened by combining with it that of a sound flogging.

UNRELIABILITY OF SURVEYS.—Even where surveys have been made, they are in many cases so unreliable that the recollection of old residents

in the vicinity, considerably stimulated, perhaps in their youth in the way described, is more reliable in determining the proper location of lost boundaries, than the retracing of old surveys. This is not surprising when the modes of surveying and the character of the instruments used are taken into consideration.

Outside of cities and larger towns, the instruments usually employed in surveying are the chain and compass. The method is to perambulate the boundary line of the tract to be surveyed, taking the magnetic bearings with the compass, and measuring with the chain the lengths of each side of the polygon forming the boundary.

IMPERFECTION OF THE COMPASS.—Now, provided the bearings thus taken were precisely measured angles from fixed parallel meridians, whose directions could always be easily ascertained, when a re-survey should be needed, no better method of noting directions could be desired. But this is far from being true. The magnetic force to which the direction assumed by the needle is due, is quite irregular in its action, changing its direction continually, backwards and forwards, even during the different hours of the day, while larger oscillations extending sometimes to many degrees of arc take place in irregular cycles, perhaps one or two centuries in duration. And this is not all. In regions containing metallic deposits, especially magnetic iron ore, very irregular and powerful local disturbances of the magnetic force arise, causing the needle to take widely different directions, even at points in close proximity to each other, thus destroying parallelism of action, and rendering the compass quite useless for ascertaining true directions.

Beside the uncertainty of the magnetic meridian, there is an incapacity of precision in the use of the compass for measuring angles. The needle must swing clear of the graduated limb, and cannot be suspended for field use, with very great delicacy. In practice it is usually impossible to read a magnetic bearing with greater precision than to the nearest 10 minutes of arc.

THEODOLITE AND SURVEYOR'S TRANSIT.—The theodolite and the surveyor's transit are instruments of far greater precision and are generally used in cities, and for more valuable farm lands, also for engineering works, roads, railroads, &c. Those in ordinary use, measure angles to single minutes.

In land surveying, the common method of using these instruments is to measure the angles formed by the sides of the bounding polygon with

each other, and sometimes, for verification, with one or more diagonals. The compass needle, usually attached, affords an approximate means of ascertaining azimuths, but with no more precision than with the ordinary compass. It is accordingly, quite as difficult to retrace obliterated boundaries with these instruments as with the compass, unless one or more well-defined lines remain for reference.

SOLAR COMPASS.—The solar compass now used in the surveys of the public lands at the West, for running out the parallel and range lines is a great improvement upon the magnetic compass in the accuracy of its azimuths if not in the precision of reading minute angles. The direction of the sun, with proper adjustment of the instrument for the latitude of the place and declination of the sun and hour of the day, affords of course, a reliable means of obtaining the true azimuth of an observed line with as much precision as the mechanical construction of the instrument permits. The use of the solar compass, however, is limited to sun-shining or slightly cloudy days, the middle portions of which, moreover, are unfavorable to accurate observations, and at best, the precision of its angular measurements is much inferior to that attainable with the transit. Nevertheless, for the preliminary surveys of wild lands, where no trigonometrical survey has been made, and where rapidity and economy are required, as in the government surveys of western lands, it is a very convenient and useful instrument.

MEASUREMENT OF DISTANCES.—The direct measurement of distances is attended with even more difficulty than that of the determination of directions. It is accomplished by the repeated application of the chain, tape or measuring rod, as nearly as possible, upon the line to be measured. Passing over inaccuracies in the length of the measuring standard arising from imperfections of construction, inequalities of temperature, changes of length by stretching, kinking, &c., the difficulties of making the direct applications are frequently quite serious. It often happens that access to all parts of the line to be measured is difficult, if not impossible. In fact, the boundaries of improved properties are generally indicated by walls, fences, hedges, ditches, etc., which occupy a considerable width upon the ground, being partly upon either side of the dividing line, upon which it therefore becomes impossible to apply the measuring standard. In such cases, it is usual to measure the opposite sides of an imaginary parallelogram, equal offsets being made at the ends of the line, and the measurements effected between the offset

points. The offset angles are quite frequently estimated by the eye, and brought as nearly as possible to right angles. Even if these angles are instrumentally measured, the operation becomes complicated, thus increasing the liability to error. Impassable obstacles along boundary lines are of frequent occurrence, and various expedients, more or less complicated, are resorted to for ascertaining the distances through them. The difficulties of measuring accurately over uneven ground, requiring a careful and laborious use of the plumb line, the avoidance of sagging or unequal stretching when the chain is used, the exact marking on the ground of the end of the measuring standard, &c., are familiar to all land surveyors.

MEASUREMENTS OF ANGLES.—It is easy to see that measurements of angles with good instruments can be performed with far greater ease and precision than the measurements of distances by the ordinary methods. Either mode of measurement is merely a determination of *ratios*. Thus, when we measure the length of a line by direct applications of a standard length upon the ground, we simply ascertain the ratio between the length of the measured line and that of the chain. So, if we measure the three angles of a triangle, we know by a simple computation, the ratio of its three sides to each other. The percentage of error in the ratios, as found by either method, is much less in the measurement of angles with good instruments; for although the distances which are compared together in this measurement, namely those marked in degrees along the limb of the instrument used, are many times smaller than those compared together in the measurement of distances upon the ground, the nicety of the mechanism and the ease of the verification by repetition admit a precision quite unattainable in actual ground measurements, except by slow and laborious processes.

TRIGONOMETRICAL SURVEY.—In the trigonometrical survey, this superiority of angle measurement is practically recognized. Convenient points are selected, at suitable distances from each other, where the angles between imaginary lines, joining adjacent points, can be measured. The whole area to be surveyed is cut up by these lines into a network of triangles, and the ratios of the sides of these triangles to each other are determined by measuring the angles between them. Then, when we ascertain the length of any one side of any one triangle, we can compute all the sides of every triangle, or the distances from point to point throughout the whole survey.

DEGREE OF ACCURACY ATTAINED.—The accuracy of the result, accordingly, depends upon the precision with which this one side or base line is measured, as well as upon the accuracy of the angle measurements. It is usual to verify the whole of the work by measuring another base line in a distant part of the network of triangles. For example, in the Coast Survey work, a base line was measured at Fire Island, on the south side of Long Island, in 1834; another in Attleborough and Sharon, Massachusetts, in 1844; and another near the village of Epping, in Columbia, Washington county, Maine, in 1857. The distance between the Fire Island and Massachusetts bases, along the axis of triangulation, was 230 miles, and between the Massachusetts and Epping bases 295 miles. The length of the Fire Island base, 8715.942 metres, or 5.415 miles, as actually measured, varies from the length, as computed from either of the other two bases, by less than 0.07 metres, or 2.75 inches; and the probable error of any computed line between these two bases is shown by careful analysis not to exceed $\frac{1}{2881000}$ of its entire length. This proportion of error to distance amounts to 0.22 inches in a mile, or a little less than 2 feet in 100 miles.

This degree of accuracy indicates the wonderful skill which has been attained in the construction and use of instruments both for the measurement of base lines and of angles. Thus, if we compare the actual distance upon the limb of a theodolite, 30 inches in diameter, which corresponds to an error of $\frac{1}{2881000}$, which we will suppose to be all thrown into one of the three angles of a single well conditioned or nearly equilateral triangle, we shall find it equivalent to about $\frac{1}{201000}$ of an inch,* being about 0.71 seconds of arc.

The percentage of error here developed is so small that it would not practically vitiate the measurement of lands even in the most valuable localities of great cities.

SUPERIORITY OF THE TRIGONOMETRICAL METHOD.—The degree of precision commonly attained in direct measurements of distances by ordinary methods falls very far short of this, and even of that attainable in angle measurements with the ordinary surveyor's theodolite or transit.

It follows that accuracy, even in common surveying, would be promoted by using the method of triangulation for ascertaining ratios, whenever practicable or convenient, in preference to the common meth-

* More exactly 0.000052 inches, for we have radius = 57.3°, nearly, in terms of arc; or, making radius = unity, $1^\circ = 57.3''$, $1' = 3438''$ and $\frac{1}{2881000} = 0.71''$ nearly. Conversely, in terms of distance, if radius = 15 inches, $\frac{1}{2881000} = 0.000052$ inches.

ods of traverse surveying. In the survey of a field, or tract of land, for example, triangulation from one or two judiciously selected and carefully measured bases would give the positions of the corners and other objects with greater accuracy and far less labor than the usual routine of perambulating the outside boundaries, which is now taught in treatises on surveying, and generally practiced by surveyors.

ADVANTAGES OF COMBINING SURVEYING WITH GEODESY.—With the facilities afforded by the Coast Survey triangulations, when carried to the tertiary stage, it is not difficult to perceive that a general method of determining the positions of points by co-ordinates might be established, and that while determinations thus made would be far more satisfactory and definite than those obtained by the ordinary methods of surveying, they would involve no more labor. Under such a system, the field work might consist of such a combination of triangulation and traverse surveying as would be found most convenient under the special circumstances.

To carry out such a system, the trigonometrical stations should be located so near together that two or more of them would be available for any subsequent local survey. If not otherwise visible, there should be convenient arrangements for the temporary erection of signals upon stations to indicate their positions, making them visible from adjacent stations.

TRANSFORMATION OF CO-ORDINATES.—While for geographical purposes, the great co-ordinate planes already described are the most suitable for reference, simplicity and convenience would be promoted by transforming this general co-ordinate system into numerous plane rectangular systems in limited local areas. Accordingly, instead of defining the position of a point by giving its terrestrial latitude and longitude, we would give its "latitude and departure," or its co-ordinates, from the zero point or origin of co-ordinates for the containing area.

For this purpose the local areas, into which the earth's surface is subdivided, should be made sufficiently small to reduce the error which would arise from considering each separate area, or plane surface, to an inconsiderable amount. Six miles square is not far from the average area of townships in the northern, middle, western and most of the southern States. The bulge of curvature, or the versed sine of half the arc of 6 miles would be about 6 feet. This would make the proportional difference between the straight and the curved distance, or between the

length of the chord and of the arc, much less than the percentage of probable error inseparable from measurements of the utmost attainable precision in actual practice. Township boundaries, therefore, would seem to afford the most convenient divisions between separate co-ordinate systems. If, however, as is the case in a few of the southern States, no smaller civil subdivisions than counties exist, these are not usually so large that the use of a single co-ordinate system over its area would involve any important error.

DIRECTION OF AXES.—The most appropriate and convenient directions for the axes would doubtless be found in meridians and perpendiculars to them, since azimuths reckoned from the meridional axis would then conform very nearly to the true astronomical azimuth. Owing to the convergence of meridians, there would be a small variation from the true azimuth increasing with the distance from the axis, but no practical error need arise from this cause. The true azimuth, if needed, is easily computed by the formula,

$$\tan. \frac{1}{2} C = \sin. L \tan. \frac{1}{2} P,$$

in which L is the middle latitude between the origin and the point where the convergence is to be computed, P the difference of longitude between the same points, and C the convergence sought. In passing from one district to another, however, a certain degree of complication arises and it becomes necessary to take the convergence into account. We may, without practical error, consider any two adjacent districts as lying in a plane produced by developing a conical surface tangent to the earth on the middle parallel of latitude between the origin of the two districts. On this plane, the convergence of the meridional axes of small districts will conform to the above formula, and the small angle of convergence measures the change of direction between the co-ordinate systems of the two districts.

PASSING FROM ONE DISTRICT TO ANOTHER.—In the passage from one district to another, four different cases of transformation of co-ordinates arise, namely :

1st. When the origin of the new system is east and north of the origin of the old system.

It will, of course, be necessary while assigning positive values to latitudes and departure in one direction to give negative values in the opposite direction; thus if north and east are to be reckoned as positive, south and west must be reckoned as negative.

In this first case, the old co-ordinates of the new origin are, accordingly, positive while the new co-ordinates of the old origin are negative. Or, if we call the old co-ordinates a and b , and the new ones a' and b' , a and b are here positive and a' and b' negative.

2d. When the new origin is west and north of the old, or a negative, b positive, a' positive and b' negative.

3d. When the new origin is west and south of the old, or a and b negative, and a' and b' positive.

4th. When the new origin is east and south of the old, or a positive, b negative, a' negative and b' positive.

For ascertaining the new co-ordinates of the old origin, the equations—

$$- a' = a \cos. \varphi + b \sin. \varphi,$$

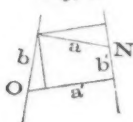
$$- b' = b \cos. \varphi - a \sin. \varphi,$$

would prove correct in all these cases, provided proper positive and negative values were given to the different terms in the equations. It is necessary to remember, however, that φ , the angle of change in direction, between the old and new axes, is positive according to trigonometrical usage when reckoned from zero around towards the left, and negative in the opposite direction, while according to the geodetic method of estimating azimuths, positive angles are reckoned around to the right.

To avoid the confusion which might arise from these different methods of estimating angles, or from assigning a negative value to φ , equations are given below for each of the four cases. Their correctness will be apparent on simple inspection of the accompanying figures, in which O and N are the old and new origins respectively :

Fig. 6.

CASE FIRST.

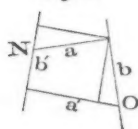


$$a' = a \cos. \varphi + b \sin. \varphi,$$

$$b' = b \cos. \varphi - a \sin. \varphi.$$

Fig. 7.

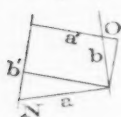
CASE SECOND.



$$a' = a \cos. \varphi + b \sin. \varphi,$$

$$b' = b \cos. \varphi - a \sin. \varphi.$$

Fig. 8.

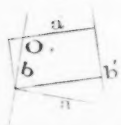


CASE THIRD.

$$a' = a \cos. \varphi - b \sin. \varphi,$$

$$b' = b \cos. \varphi + a \sin. \varphi.$$

Fig. 9.



CASE FOURTH.

$$a' = a \cos. \varphi - b \sin. \varphi,$$

$$b' = b \cos. \varphi + a \sin. \varphi.$$

In these equations the negative sign is omitted before a' and b' , but we must remember that they are always estimated in directions opposite to those of a and b .

Since φ is a very small angle, $\cos. \varphi$ approximates closely to unity, and it appears by these equations that a' is greater than a , and b' is less than b when the new origin is farther *north* than the old, while a' is less than a , and b' greater than b when the new origin is farther *south* than the old.

Having found the new ordinates of the old origin, that is of O referred to N , the ordinates of any point referred to the new origin may be computed from its old ordinates by the equations :

$$x = x' \cos. \varphi - y' \sin. \varphi + a$$

$$y = x' \sin. \varphi + y' \cos. \varphi + b;$$

in which x = departure of any point as referred to N ,

x' = departure of the same point as referred to O ,

y = difference in latitude of any point as referred to N ,

y' = difference in latitude of the same point as referred to O ,

a = departure of the origin of O as referred to N ,

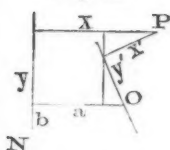
b = difference in latitude of the origin of O as referred to N ,

φ = the angle of convergence of the meridional axes of N and O .

These are the formulæ for passing from one system of rectangular co-ordinates to another in the same plane.

In these equations, we may consider north and east to be the positive directions, as before, the opposite directions being negative. Also $\sin. \varphi$ is positive when the change of axial direction is towards the left, and negative when in the opposite direction ; that is, positive when N is farther east than O , and negative when farther west. $\cos. \varphi$ will always be positive, since φ is always either in the first or fourth quadrants.

Fig. 10.



In Fig. 10, the point P, of which x and y are the new co-ordinates, is east and north of the axes passing through O, making x and y greater than a and b respectively, and all the terms of both equations have, accordingly, positive values. But if x , x' or a be estimated towards the west, its sign must be reversed when particular values are substituted in the equations; likewise, if y , y' or b , have a southern direction, its sign must be reversed.

CO-ORDINATES IN A SINGLE DISTRICT.—It is hardly necessary to say that when the azimuth and distance are given from a point whose co-ordinates are known to any other point, the co-ordinates of the latter are found by multiplying the given distance by the sine and co-sine of the azimuth, the first product giving the departure and the other the difference of latitude.

VERIFICATIONS.—Before finally establishing the co-ordinates of a survey its accuracy should be tested in the most rigid manner, both as regards the instrumental observations and the computations. The computations are easily verified by working to the same point from different directions. Some methods of verifying the field work are indicated in the accompanying plates and explanations. Others will suggest themselves to the surveyor under different circumstances.

ILLUSTRATIONS OF NOTATION.—Plates XII, XIII and XIV, exhibit the sort of notation which may be employed under the system proposed. Instead of magnetic bearings or angles written between lines, azimuths are given, which are estimated around to the right from zero at due north to 360° or due north again. From these azimuths, angles around to the right are easily found by subtracting the first azimuth from the second, adding 360° to the latter if zero comes between. (More complete explanations are given on pages 102 and following.)

The co-ordinates are here given in feet, but metres, chains or any other standard units may be used in the same way. The letters N. E. S. W. indicate the directions for the origin, north, east, south or west. Points upon the division lines between two adjacent districts have their co-ordinates given in both. For railroad surveys, it would be found convenient in plotting to have the co-ordinates of tangent points and centres of curves given, even though the latter should not appear upon the plan. Other convenient details of notation will suggest themselves to engineers, and indeed the plates are only intended to present to the eye the

general features of the method proposed. A great variety of cases will arise in practice, many of them requiring special treatment.

COMPUTATION OF AREAS.—Areas are computed under this system with special facility and certainty, the method being the common one of double latitudes and departures. This method is prescribed by law in the State of Ohio, for calculating areas of farming lands and for testing the accuracy of surveys made with the surveyor's compass.

DESCRIPTIONS FOR CONVEYANCES.—For definite and accurate descriptions of land in conveyances, it does not seem possible to devise a more precise and certain method than that of co-ordinates from geodetically determined reference points or origins. The present loose and indefinite descriptions in conveyances, upon which the tenure of a large part of the real estate of the country now depends, are disgracefully uncertain and frequently lead to excessive expenses of unnecessary litigation, and sometimes to costly errors of misplaced constructions.

CONVENIENCE IN CONSTRUCTING MAPS.—In the construction of maps and plans, co-ordinate determinations will be found especially convenient. After completing the survey of any portion of a district it is easy to place it in its proper position upon the map of the entire district with the certainty that other portions subsequently surveyed will fit into their proper places without the perplexity and the distortions frequently accompanying the attempts to unite two or more independent surveys made under the methods in common use.

SUMMARY OF ADVANTAGES.—A few of the advantages which may be expected to follow the general adoption of the co-ordinate method of surveying may be summed up as follows :

First.—The attainment of the highest practicable degree of accuracy, as well in smaller local surveys as in more extended operations. The units of measurement which form the basis of the United States Coast Survey have been most carefully compared with those of the entire civilized world, and with the dimensions of the earth itself, and are verified to a degree of precision beyond which the present attainments of scientific skill have not passed.

Second.—Extreme simplicity of notation with ease and convenience of field work and computation.

Third.—Facility in graphic representation.

Fourth.—Absolute certainty of locations in descriptions for conveyances, and consequent removal of a fruitful cause of litigation and trouble.

ALTITUDES.—No change is proposed in the existing methods of determining the third ordinate or altitude. The most convenient mode of fixing this ordinate, pre-supposes that the form of the earth's surface, or of that surface which would be presented if the irregular surface of the land were razed to the level of the sea, has been accurately determined by geodetic operations, so that in ordinary surveying we have only to ascertain the heights of our points of survey above this imaginary level. This is done by using the spirit level, by measuring vertical angles and by barometrical observations. The first method admits the greatest degree of precision under ordinary circumstances, and is almost exclusively used for engineering purposes.

EXPLANATION OF THE PLATES.—Plate XII, represents a tract of land lying partly in Pomfret and partly in Weston, two adjacent towns. One side is bounded by a lake, and a road passes through the tract. Several of its corners are visible from the point A. The point B, where the town line crosses the west side of the road, is one of the stations fixed by the preliminary trigonometrical survey. All the co-ordinates upon this plan have been determined by computing the latitudes and departures, directly or indirectly, from this point. The convergence of the axial meridians in the two towns is $31''$, and the co-ordinates of the origin in Weston referred to Pomfret are,

$$a = 3\,513.27 \text{ E, and } b = 15\,217.81 \text{ S.}$$

From these, we can compute the ordinates of the Pomfret origin referred to Weston, by the formulæ for Case Fourth (page 100) :

$$a' = a \cos. \varphi - b \sin. \varphi,$$

$$b' = b \cos. \varphi + a \sin. \varphi, \quad \text{or}$$

substituting values ($\cos. 31'' = 1$ log. $\sin. 31'' = 6.17\,693\,65$.)

$$a' = 3\,513.27 - 15\,217.81 \sin. 31'' = 3\,510.983$$

$$b' = 15\,217.81 + 3\,513.27 \sin. 31'' = 15\,218.338.$$

Reversing the directions,

$$a = 3\,510.983 \text{ W. } b = 15\,218.338 \text{ N.}$$

Equations for passing from Pomfret into Weston.—The general formulæ (page 100) are :

$$x = x' \cos. \varphi - y' \sin. \varphi + a$$

$$y = x' \sin. \varphi + y' \cos. \varphi + b.$$

In this case $\varphi = + 31''$; (in passing eastward from Pomfret to Weston the axes swing around to the left, and the angle of change is

positive, according to trigonometrical usage); $a = 3\,510.983$, and $b = 15\,218.328$. Substituting values, the equations become :

$$x = x' - y \sin. 31'' - 3\,510.983$$

$$y = x' \sin. 31'' + y' + 15\,218.338.$$

Equations for passing from Weston to Pomfret.—In this case, $\varphi = -31''$ (reckoned to the right), $a = 3\,513.27$ and $b = 15\,217.81$; substituting values :

$$x = x' + y' \sin. 31'' + 3\,513.27$$

$$y = -x' \sin. 31'' + y' - 15\,217.81.$$

Double pairs of co-ordinates are given along the town line, and either pair may be computed from the other by using these equations. The accuracy both of the equations and the computations are verified by reversing the method.

Verification of the Survey.—At the point C, where three trigonometrical stations can be seen, azimuths were taken to each and the co-ordinates of C computed by the "three point problem." They are 859.36 W, and 7 073.59 N, from the origin of Weston. The azimuth from C, to the south-east corner of the tract is $138^{\circ} 41'$, and the distance 63 feet, giving the co-ordinates of C, 859.41 W., 7 073.51 N. The degree of accuracy here indicated would probably be sufficient under ordinary circumstances.

PLATE XIII.—We have here an operation frequently performed by land surveyors, namely, the division of a tract into house lots. Streets and alleys are projected parallel with Main street, making the lots 100 feet in depth and the streets 40 feet, and alleys 15 feet in width.

As in Plate XII., a town line bound-stone is also a trigonometrical station. This bound is at A, on the south side of East street, and its co-ordinates, as determined by the trigonometrical survey, are given in the drawing. At this point we obtain an azimuth to a station in Waterford, lying southeast of and visible from A, and from this azimuth all the others in the plan are obtained.

A part of the survey extends into the adjacent town of Hollis, and we have the problem of passing from one co-ordinate system to another in a little different form from that of Plate XII. In this case, the ordinates of the origin in Waterford referred to Hollis, are :

$$a = 42\,361.47 \text{ E, } b = 9\,312.13 \text{ N, and } \varphi = 6^{\circ} 14.18'.$$

By the formulæ for Case Third :

$$a' = a \cos. \varphi - b \sin. \varphi$$

$$b' = b \cos. \varphi + a \sin. \varphi.$$

Substituting values ($\cos. 6' 14.8'' = 1$ log. $\sin. 6' 14.8'' = 7.258\ 390\ 6$

$$a' = 42\ 361.47 - 9\ 312.13 \sin. 6' 14.8'' = 42\ 344.588$$

$$b' = 9\ 312.13 + 42\ 361.47 \sin. 6' 14.8'' = 9\ 388.948 ;$$

or with reversed directions, 42 344.588 W., 9 388.948 S.

Equations for passing from Waterford to Hollis.—In the general formulæ,

$$x = x' \cos. \varphi - y' \sin. \varphi + a$$

$$y = x' \sin. \varphi + y' \cos. \varphi + b,$$

we have $\varphi = - 6'.14.8''$ and the equations become :

$$x = x' + y' \sin. 6' 14.8'' + 42\ 361.47$$

$$y = - x' \sin. 6' 14.8'' + 7+9\ 312.13.$$

Equations for passing from Hollis to Waterford.—Here $\varphi = 6' 14.8''$ and the equations become :

$$x = x' - y' \sin. 6' 14.8'' - 42\ 344.588$$

$$y = + y' \sin. 6' 14.8'' + y' - 9\ 388.948.$$

Verification of the Survey.—The location of the town line boundary B, on the west side of Main street, has been determined by the trigonometrical survey, and affords the means of verifying the position of north-western corner of the tract. The point C outside of the tract, to the east, and not shown on the plan, is visible from some of the easterly lot corners of the tract, from which azimuths are taken to it, enabling us to compute its co-ordinates from the present survey, which is thereby verified.

PLATE XIV.—In running railway surveys, every opportunity should be taken to connect with the trigonometrical stations which become accessible near the line, so as to verify its direction and the position of the stakes or stations. The methods of doing this by triangulation and otherwise are simple, and will readily suggest themselves to the engineer. This plate illustrates the passage of a railroad survey across a town line, passing from one system of ordinates to another.

The ordinates of the origin in Dexter referred to Elliot, are :

$$28\ 243.13\ E.\ 31\ 497.21\ N.$$

and the convergence is $4' 10''$.

By the equations given for Case First (page 99) we find the ordinates of Elliot referred to Dexter to be

$$(\cos. 4' 10'' = 1 \log. \sin. 4' 10'' = 7.081\,937\,6.)$$

$$a = 28\,243.13 + 31\,497.21 \sin. 4' 10'' = 28\,281.134$$

$$b = 31\,497.21 - 28\,243.13 \sin. 4' 10'' = 31\,463.103$$

or reversing directions, 28 281.134 W., and 31 463.045 S.

Equations for passing from Dexter to Elliot.—The general formulæ are

$$x = x' \cos. \varphi - y' \sin. \varphi + a$$

$$y = x' \sin. \varphi + y' \cos. \varphi + b.$$

In this case $\varphi = 4' 10''$, $a = -28\,281.134$ and $b = -31\,463.045$; and the equations become :

$$x = x' - y' (-\sin. 4' 10'') - 28\,281.134$$

$$\text{or } x = x' + y' \sin. 4' 10'' - 28\,281.134,$$

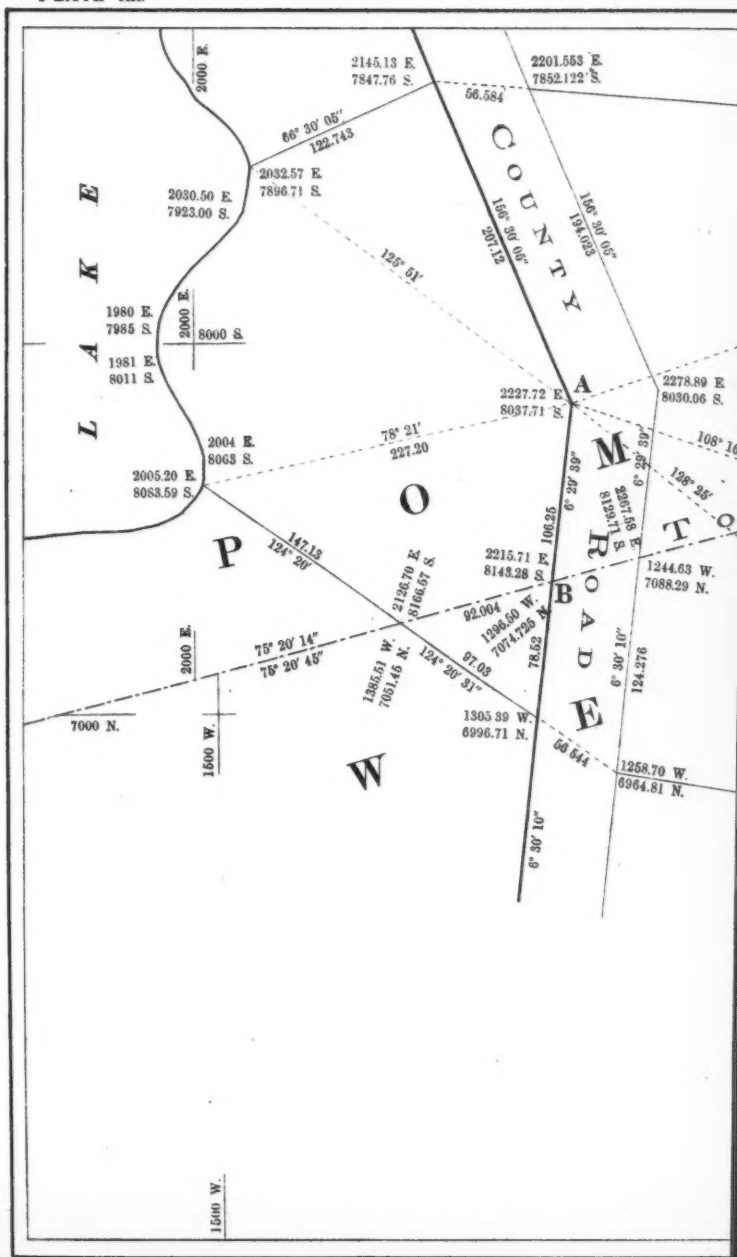
$$\text{and } y = -x' \sin. 4' 10'' + y' - 31\,463.045.$$

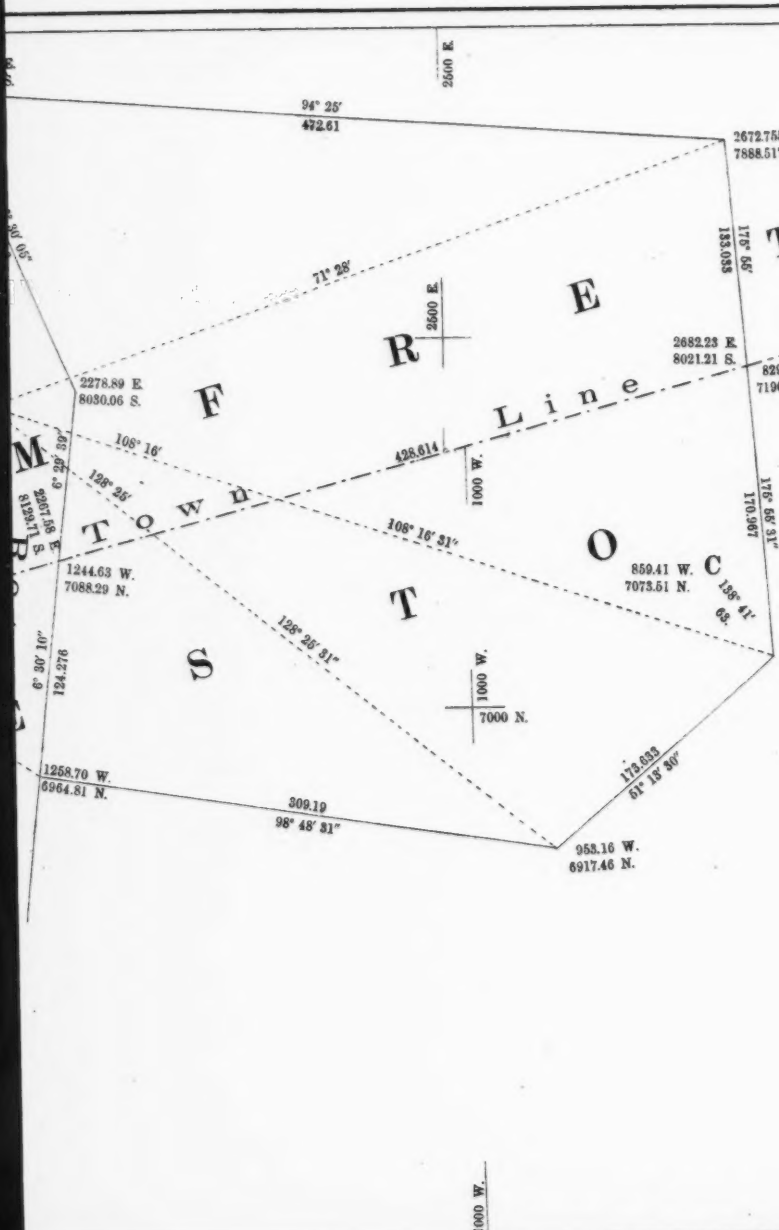
Equations for passing from Elliot to Dexter.—Here $\varphi = 4' 10''$, $a = 28\,243.13$, $\varphi = 31\,497.21$, and the equations become :

$$x = x' - y' \sin. 4' 10'' + 28\,243.13$$

$$y = x' \sin. 4' 10'' + y' + 31\,497.21.$$

PLATE XII.





2672.755 E.
7888.517 S.

175 56
175 56

T

8000 S.

829.96 W.
7196.72 N.

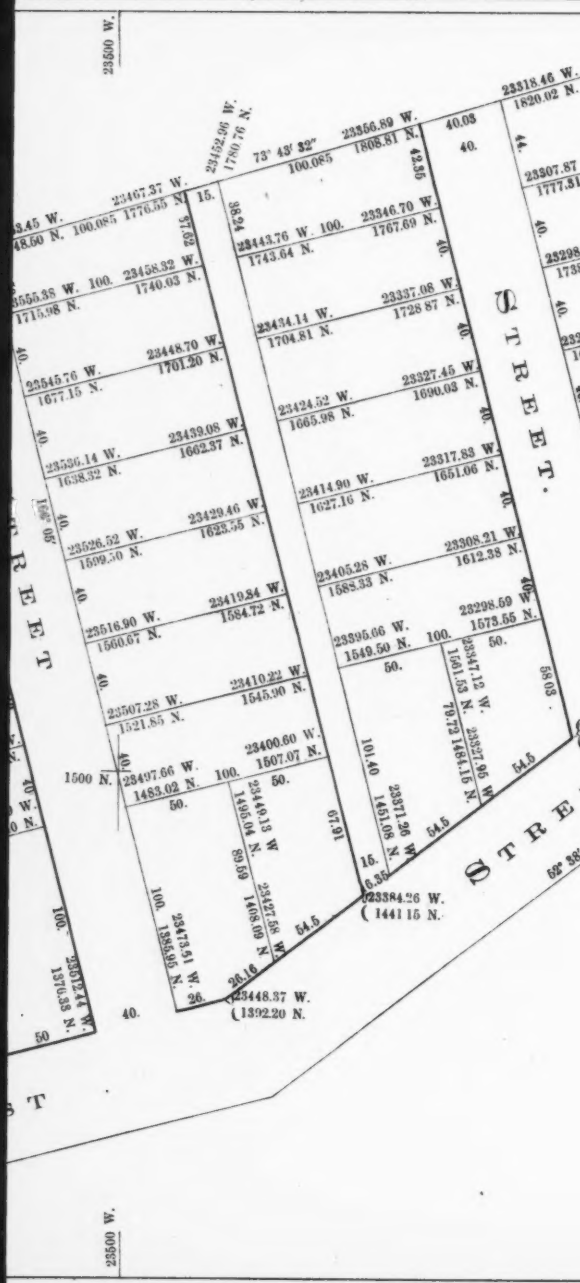
175 56
175 56

N

817.82 W.
7026.19 N.

7000 N.





23500 W.

23467.37 W.
48.60 N. 100.085 1776.55 N.

23458.32 W.
1715.98 N. 1740.03 N.

23446.70 W.
1677.15 N. 1701.20 N.

23439.08 W.
1638.32 N. 1662.37 N.

23429.46 W.
1609.50 N. 1623.55 N.

23419.84 W.
1560.67 N. 1584.72 N.

23410.22 W.
1521.85 N. 1545.90 N.

23400.60 W.
1483.02 N. 1507.07 N.

23473.51 W.
100. 1386.96 N.

23448.37 W.
1392.20 N.

23443.76 W. 100. 1743.64 N.

23434.14 W. 1704.81 N.

23424.52 W. 1665.98 N.

23337.08 W. 1728.87 N.

23327.45 W. 1690.03 N.

23317.83 W. 1651.06 N.

23356.89 W. 1808.81 N.

23346.70 W. 1767.69 N.

23337.08 W. 1728.87 N.

23327.45 W. 1690.03 N.

23317.83 W. 1651.06 N.

23308.21 W. 1612.38 N.

23298.59 W. 1573.55 N.

23284.12 W. 1534.72 N.

23271.26 W. 1495.04 N.

23257.96 W. 1455.04 N.

23244.26 W. 1415.15 N.

23230.78 W. 1375.15 N.

23217.40 W. 1335.15 N.

23204.02 W. 1295.15 N.

23190.64 W. 1255.15 N.

23177.26 W. 1215.15 N.

23163.88 W. 1175.15 N.

23150.50 W. 1135.15 N.

23318.46 W. 1820.02 N.

23307.87 W. 1777.51 N.

23298.59 W. 1738.00 N.

23284.12 W. 1698.00 N.

23271.26 W. 1658.00 N.

23257.96 W. 1618.00 N.

23244.26 W. 1578.00 N.

23230.78 W. 1538.00 N.

23217.40 W. 1498.00 N.

23204.02 W. 1458.00 N.

23190.64 W. 1418.00 N.

23177.26 W. 1378.00 N.

23163.88 W. 1338.00 N.

23150.50 W. 1298.00 N.

23137.12 W. 1258.00 N.

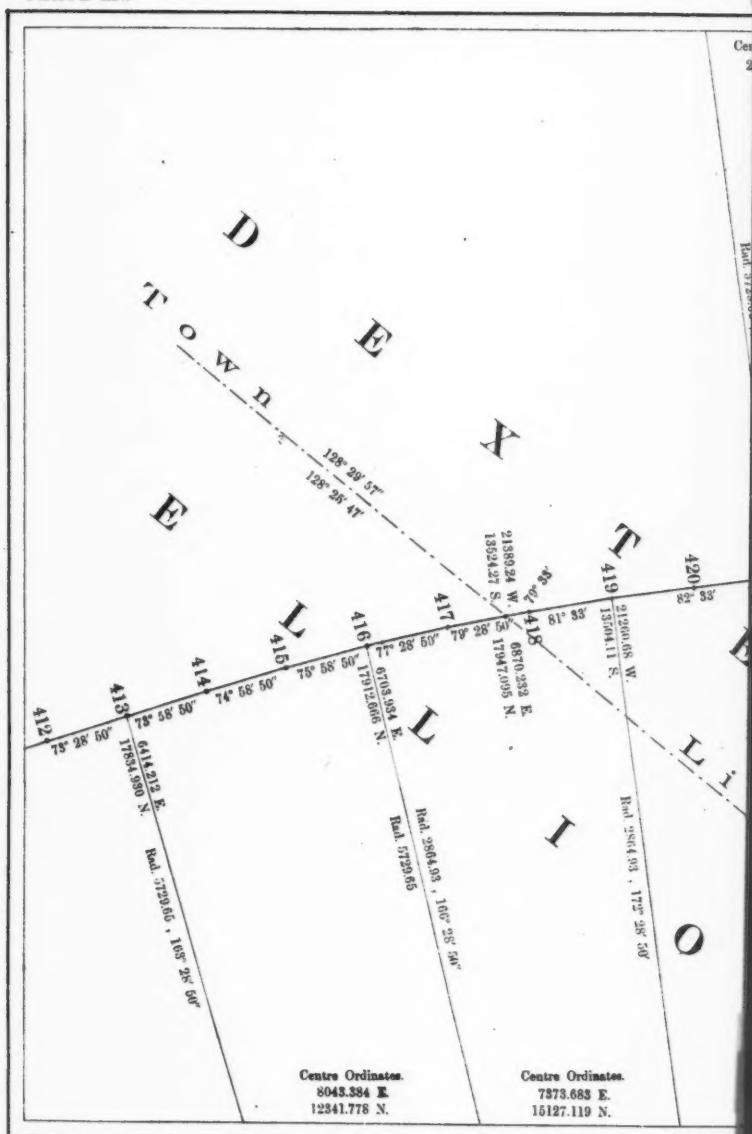
23123.74 W. 1218.00 N.

23110.36 W. 1178.00 N.

23096.98 W. 1138.00 N.



PLATE XIV.



21805.277 W.
7796.900 S.

21178.154 W.
11564.631 S.

22386.959 W.
7941.381 S.

Red. 57.9.65, 169° 38'

$$\begin{array}{r} 20385.079 \text{ W.} \\ 13306.988 \text{ S.} \\ \hline \end{array}$$

88

427

31° 0'

26

425

124

23

22

42

420

43260.68 W

Ref. 2804.03, 172° 28' 00"

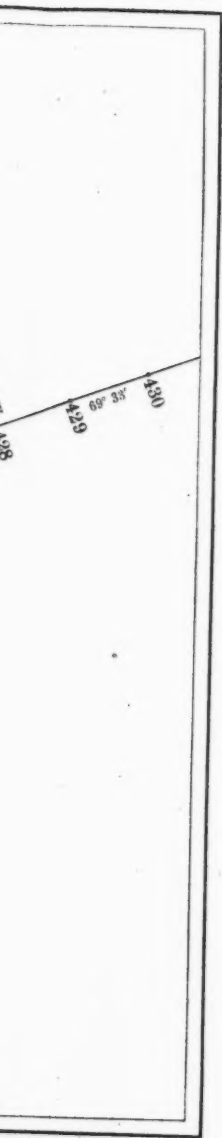
E

R

Line



T



AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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CXXXIX.

THE CONSUMPTION AND WASTE OF WATER

DELIVERED BY PUBLIC WORKS.

A Paper by JAMES H. HARLOW, C. E., Member of the Society.

READ MARCH 1ST, 1876.

The supply of water to cities and towns for domestic and manufacturing purposes, has within the last ten years taken a prominent place as an engineering problem, and a very important question to be decided before accepting or rejecting the different schemes of supply presented in any particular case, is the probable amount of consumption.

To show what the consumption has been in different cities at different times, the table on the following pages is presented. The data have been principally obtained from the annual reports published by the city in whose interest the works have been run. The population of 1850, 1860 and 1870 is taken from the United States census, and for the years between, it has been estimated at a uniform rate of increase. This method has been chosen because it will give a fair comparison between different places. The figures given do not in all cases agree with those in the annual reports, on account of the difference in the estimated population. There is a tendency to overestimate the population, and in fact the population for several places has been stated some thousands greater than that given by the United States census, several years later.

STATISTICS RELATING TO THE CONSUMPTION OF WATER.

Year.	Ratio of lakere to population.	Inhabitants per mile of pipe.	Average number of gallons, per day.		Ratio of lakere to population.	Inhabitants per mile of pipe.	Average number of gallons, per day.		Ratio of lakere to population.	Inhabitants per mile of pipe.	Average number of gallons, per day.			
			Per capita.	Per taker.			Per capita.	Per taker.			Per capita.	Per taker.		
BOSTON, MASS.					DETROIT, MICH.					PHILADELPHIA, PA.				
1849	27.6		
1850	42.7		
1851	48.9		
1852	56.3	28.5		
1853	0.122	57.6	470.1	0.152	856	29.4	194.	The different works were consolidated in 1854.					
1854	0.126	1.421	65.1	515.9	0.158	790	35.2	223.2	1.778	24.8		
1855	0.128	1.425	66.2	517.4	0.164	699	46.3	281.6	1.797	27.4		
1856	0.13	1.427	75.2	579.1	0.167	757	55.5	332.4	1.787	50.9		
1857	0.131	1.423	77.2	589.1	0.157	873	46.3	306.9	1.702	33.9		
1858	0.133	1.432	76.1	573.1	0.159	876	45.9	303.9	1.739	35.		
1859	0.134	1.385	76.	566.1	0.152	768	48.1	315.4	1.670	35.7		
1860	0.136	1.371	95.9	708.9	0.152	748	52.2	342.9	1.624	36.1		
1861	0.139	1.462	98.8	713.7	0.154	719	53.3	344.	1.598	36.1		
1862	0.138	1.431	87.1	631.4	0.155	708	58.3	374.7	1.587	37.		
1863	0.135	1.457	82.3	610.9	714	58.5	369.	1.571	43.5		
1864	0.132	1.495	81.7	616.8	0.157	717	56.6	359.5	1.571	41.8		
1865	0.13	1.536	59.7	460.6	0.161	711	55.4	337.	1.563	48.9		
1866	0.127	1.565	56.9	440.6	0.161	723	59.6	360.6	1.542	46.1		
1867	0.125	60.	482.7	0.177	730	63.8	360.2	1.510	46.4		
1868	0.127	63.6	495.9	0.169	693	67.2	396.	1.485	50.3		
1869	0.13	1.425	62.2	478.9	0.172	655	61.	354.1	1.435	51.3		
1870	0.144	1.288	59.9	415.3	0.171	674	64.2	372.6	1.377	54.4		
1871	0.149	1.184	53.4	360.2	0.172	667	73.	355.9	1.318	54.		
1872	0.152	1.132	56.1	370.2	0.174	667	82.6	474.	1.271	51.6		
1873	0.152	1.059	64.2	421.3	0.176	639	90.1	515.	1.206	55.2		
1874	0.187	633	87.3	365.8	1.142	55.5		
CLEVELAND, O.					CHICAGO, ILL.					JERSEY CITY, N. J.				
1857	0.112	2.154	12.4	110.7	62.8		
1858	0.126	2.358	11.8	93.4	0.051	1.259	32.8	641.	70.		
1859	0.148	1.690	13.5	91.3	0.055	1.175	38.8	702.	74.7		

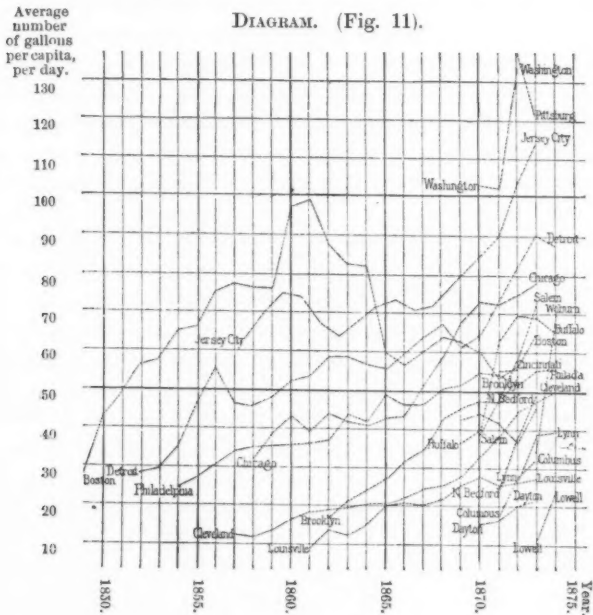
STATISTICS RELATING TO THE CONSUMPTION OF WATER.—(Concluded.)

Year.	Ratio of takers to population.	Inhabitants per mile of pipe.	Average number of gallons, per day.		Ratio of takers to population.	Inhabitants per mile of pipe.	Average number of gallons, per day.		Ratio of takers to population.	Inhabitants per mile of pipe.	Average number of gallons, per day.	
			Per capita.	Per taker.			Per capita.	Per taker.			Per capita.	Per taker.
CAMBRIDGE, MASS.												
The city bought out a private company in 1865.												
1869	42.2
1870	43.9
1871	627	41.6	1 188	17.2	1 577	54.9
1872	0.221	617	36.6	165.9	0.037	1 030	27.6	748.6	1 565	55.3
1873	0.243	585	45.1	185.3	0.039	1 107	32.4	823.8
1874	46.5
COLUMBUS, O.												
CINCINNATI, O.												
DAYTON, O.												
LOWELL, MASS.												
LYNN, MASS.												
1870	1 632	15.5
1871	1 613	16.6
1872	0.028	1 529	20.2	704.5	0.094	717	26.7	282.5
1873	0.031	1 608	21.8	706.2	0.095	1 295	10.9	116.	0.105	695	38.5	367.6
1874	0.141	1 225	24.1	112.4	0.129	706	39.3	301.5
NEW BEDFORD, MASS.												
PITTSBURGH, PA.												
SALEM, MASS.												
1870	38.5
1871	1 023	24.4	47.2
1872	916	38.	137.6	58.8
1873	857	48.9	120.6	543	72.4
WASHINGTON, D. C.												
WOBURN, MASS.												
1870	102.9
1871	101.8
1872	133.6
1873	0.087	372	33.8	775.
1874	0.123	399	69.7	561.

A "taker" in the table given, has been assumed to mean essentially the same as each of the following, viz: a taker in Boston, a tap in

Brooklyn, a service in Buffalo, a house service in Chicago, a consumer in Cleveland and a service in Louisville. The only cases in which this would seem doubtful are—Buffalo* and Cleveland. The table covers as long a period as possible from the notes in hand.†

That we may have an easy means of comparison between the consumption of any city in different years, and the different cities with each other, the following diagram is given, illustrating the foregoing table so far as it relates to the average consumption *per capita* per day.



By examination it will be seen, that with the exception of Boston, the tendency of the consumption per capita has been upwards, and this is due in some measure to the ratio of takers to population, but that this does not hold good, except to some extent, may be seen on further examination.

In cases where the ratio of takers to population is the same, we find the following examples :

* In answer to a letter on the subject, the engineer at Buffalo says : " Your table is correct."

† Up to May, 1875.

{ Boston, 1872, 1873.	Ratio 15.2 per cent.	Consumption	56.1 & 64.2 gallons per capita.
{ Detroit, 1853, 1859, 1860.	" 15.2 "	" "	29.4, 48.1, 52.2 "
{ Boston, 1854.	" 12.6 "	" "	65.1 "
{ Cleveland, 1858.	" 12.6 "	" "	11.8 "
{ Louisville, 1868.	" 3.3 "	" "	21.9 "
{ Buffalo, 1869.	" 3.3 "	" "	36.5 "
{ Chicago, 1861.	" 5.6 "	" "	39.4 "
{ Brooklyn, 1863.	" 5.6 "	" "	21.2 "

Here we see that cities having the same ratio of takers to population not only do not agree with each other in the amount consumed, but they do not agree with themselves. It is probable that the temperature has some effect on the consumption, as extremes of heat and cold are known to have caused greater draught of water. None of the above examples have occurred in the same year, and even if occurring, the places are so distant from each other that the temperature would not be likely to be the same.

It has been customary in determining the probable quantity needed, to estimate the population of the place under consideration, and then to assume some number of gallons per capita. This is done under the assumption that because a certain number of gallons per capita is necessary in one place it will be in another, and because it is convenient and approximately correct.

This would be nearly correct if the two places compared were similar, but that they are not similar will be seen by examination of the table, where we find that for 1873 there were used in the different places the following numbers of gallons per capita, per day, viz. : 10.9, 21.8, 27.2, 32.4, 33.8, 38.5, 45.1, 47.1, 48.9, 55.2, 64.2, 68.9, 72.4, 77.7, 90.1, 113.4, and 120.5, no two places using the same quantity, and the nearest any two come to each other is 1.4 gallons, the greatest difference being 109.6 gallons. The average quantity for these seventeen places is 56.9 gallons per capita per day, and nine of the seventeen use less, and eight more than the average. One place uses 45.5 gallons less, and one place 64.1 gallons more than the average.

If we should take the above places in the order of their consumption, and place small figures to show the relative rank in population, we should have

10.9,	21.8,	27.2,	32.4,	33.8,	38.5,	45.1,	47.1,	48.9,	55.2,	64.2,
11	12	5	14	17	13	10	7	16	1	3
68.9,	72.4,	77.7,	90.1,	113.4,	120.5.					
4	15	2	9	8	6					

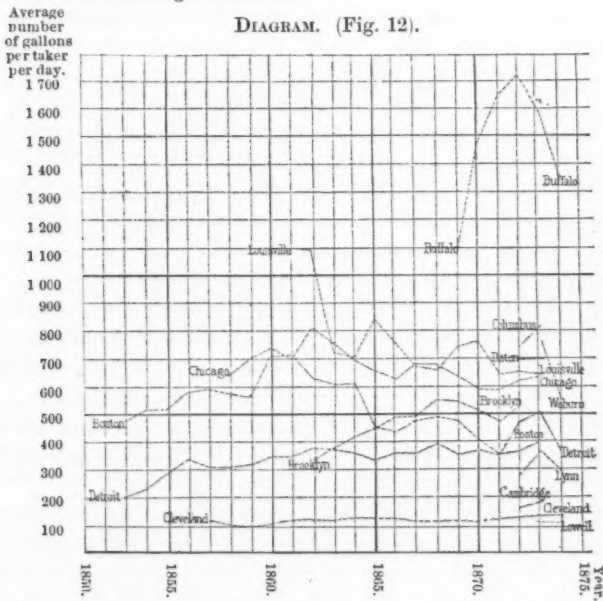
The city ranking first in population is within 1.7 gallons of the average consumption, and with two excep-

tions, the cities having the largest population use the most water per capita. This may be accounted for, in some measure, because in estimating the number of gallons used the United States census is taken, but this does not always give the number of persons to whom the water should be charged. Those persons having only their places of business in town, and those stopping at hotels, are users of water, and not being counted bring the rate per capita high. The ratio of takers to population is usually higher in large cities than in small.

Although the ratio of takers to population and the temperature seem to or may have great influence on the amount consumed, there must be some other cause for this difference. With the same population, it may be not unreasonable to suppose that there will be an average number of large and small takers, and on this assumption let us see what the average consumption is per taker.

By reference to the table, the exact figures can be seen, but for convenience the following is inserted :

DIAGRAM. (Fig. 12).



It will be seen upon examination of these diagrams that the curves drawn do not in general correspond. For example, Louisville starts with a low rate of consumption per capita, and makes a tolerably

regular rise ; but she starts with a high rate of consumption per taker, and after a very irregular course falls to a fair rate of consumption, although still high. Cleveland starts low, and with quite a regular ascent ends at about an average rate of consumption per capita ; but in the consumption per taker she makes an approximately straight line, showing that each of the different uses to which the water is put must increase in a uniform ratio, and that the waste, if any, must be about the same each year. Detroit is rather high per capita, but makes a fair line per taker, except in 1872. Buffalo is not only high per capita, but also very high per taker. Boston makes very nearly the same line per capita and per taker. Lowell and Woburn increase their rate of consumption very fast, and the superintendents complain of the great waste, but the consumption per taker is in the opposite direction, showing that other points besides the per capita rate should be examined before complaining of waste. The continual complaint of waste has of itself but little effect in reducing the consumption.

In 1873, we find the amount consumed per taker was respectively
 116, 137.7, 185.3, 367.6, 421.3, 515, 643, 651.6, 706.2, 775,
_{8 5 7 10 2 6 1 4 9 12}
 823.8 and 1600 ; the small figures showing the rank in population.
_{11 3}

The average consumption for these twelve cities is 578.5 gallons per taker per day, and the extreme differences are 462.5 gallons less, and 1 021.5 gallons more than the average. It will be seen that, in general the large cities use less per taker than the average, being the reverse of the case when reckoned per capita.

The purpose to which the water is put has its effect on the amount consumed per taker or per capita. But before considering what is done with the water, reference is made to the tables (pages 115 and following), showing the amount used under various conditions where the water has been measured.

In estimating the quantity required, the purpose for which the water is to be used should be taken into account. It certainly is undesirable to diminish the necessary quantity used for valuable purposes, while it is quite desirable to reduce all waste to its lowest minimum.

By examination of these tables it will be seen that 25 gallons per capita per day is a liberal allowance for all strictly domestic purposes, including baths and water-closets. The water pumped is used for domestic and other industrial or business purposes, and some portion, large or small, wasted.

Number.	Population.	Supply in gallons per capita per day.	Remarks.
1	5.	No. 1 came under the writer's observation, and was the amount furnished those persons whose wells were drained by the work in the vicinity.
2	2 134	7.9	
3	2 285	16.4	
4	2 574	23.	
5	1 540	16.	
6	967	17.3	Nos. 2 to 15, inclusive are fourteen of the poorer districts in Liverpool, experimented on by Geo. F. Deacon, Water Engineer, and taken from his report to the authorities. I cannot find that any of the modern conveniences except water closets were in use.
7	1 534	13.8	
8	2 570	20.7	
9	827	16.2	
10	1 824	12.3	
11	1 826	12.9	No. 16 was the amount delivered to Boston during the repairing of a break in pipes crossing Charles river. All except domestic and a few of the smaller manufactories, were shut off.
12	5 794	15.4	No. 17 was the amount used by the President of the Boston Water Board in his house.
13	809	21.1	No. 18 was average of the amount consumed in one year by the members of the Boston Water Board.
14	3 399	15.3	Nos. 16, 17 and 18, probably, had all the modern conveniences.
15	838	12.9	There is no doubt, that by the use of meters, a close approximation to these figures may be attained.
16	173 280	20.2	
17	14.	
18	24.9	

Number.	Users.	Number on which the average has been obtained.	Yearly consumption in gallons.*			Average daily consumption.	Number to whom 25 galls. per head might be delivered.
			Largest.	Least.	Average.		
19	Drinking fountains.....	about 1 000	40
20	Hotels, first class.....	11	10 993 033	3 174 700	6 050 294	16 576	663
21	" second class.....	49	2 627 660	13 033	274 927	753	30
22	Railroad companies	8	33 466 700	3 405 700	12 257 906	33 583	1 343
23	Gas light companies.....	1	47 092 933	129 022	5 161
24	" " " "	4	1 492 400	264 633	940 033	2 575	103
25	Sugar refineries.....	6	38 985 100	3 617 400	15 130 400	41 439	1 657
26	Rolling mill.....	1	16 606 733	45 438	1 820
27	Iron works.....	1	21 870 100	59 919	2 397

* From Report of Boston Water Works for 1873.—The Report for 1874 will contain a table similar to this, but more in detail, giving the number of gallons each consumer uses where meters are attached.

(Continued.)

Number.	Users.	Number on which the average has been obtained.	Yearly consumption in gallons.			Average daily Consumption.	Number to whom 25 gallons per head might be delivered.
			Largest.	Least.	Average.		
28	Brewery.....	13	9 213 000	499 400	2 570 433	7 042	282
29	Beer factories.....	4	819 566	316 933	598 667	1 640	66
30	Buildings (business)....	90	2 982 900	43 730	5 743 667	15 736	629
31	Hospitals.....	4	8 275 633	756 500	4 008 600	10 982	439
32	Benevolent homes.....	6	926 200	51 233	454 170	1 244	50
33	Club houses.....	3	1 317 000	277 933	746 800	2 046	82
34	Halls.....	3	1 325 566	733 533	979 943	2 684	107
35	Theatres.....	3	725 166	151 866	372 077	1 019	41
36	Markets.....	7	1 034 500	105 666	581 953	1 59	63
37	Colleges.....	2	441 200	354 633	397 920	1 090	43
38	Boarding houses.....	19	931 200	145 666	344 000	942	37
39	Model ".....	11	783 566	155 666	428 770	1 174	47
40	Factories.....	58	1 673 266	18 900	607 700	1 665	66
41	Machine companies.....	24	3 077 133	12 300	814 770	2 232	89
42	Foundries.....	8	3 548 500	21 500	831 780	2 279	91
43	Boilermaker.....	1	424 170	1 162	46
44	Oil works.....	5	6 546 433	151 633	1 880 800	5 153	206
45	Marble works.....	8	5 016 333	449 166	2 592 850	7 104	284
46	Stone yard.....	1	2 217 400	6 075	243
47	Vinegar works.....	1	192 733	528	21
48	Pickle ".....	5	684 133	259 166	387 033	1 060	42
49	Salt ".....	1	282 700	774	36
50	Confectionery.....	2	829 866	554 266	692 066	1 896	76
51	Restaurants.....	15	1 613 133	21 033	671 422	1 839	73
52	Saloons.....	3	1 087 500	252 033	645 044	1 781	71
53	Distilleries.....	3	2 836 366	378 333	1 504 200	4 121	165
54	Printing.....	1	912 600	2 500	100
55	Photographer.....	1	378 333	1 036	41
56	Fire brick.....	1	293 433	804	32
57	Fertilizers.....	3	423 100	286 166	339 740	931	37
58	Baths.....	3	2 597 200	846 300	1 444 600	3 958	158
59	Chemicals.....	4	7 861 266	56 433	2 737 380	7 500	300
60	Extracts.....	1	1 479 870	4 056	162

(Concluded.)

Number.	Users.	Number on which the average has been obtained.	Yearly consumption in Gallons.			Average daily Consumption.	Number to whom 25 gallons per head might be delivered.
			Largest.	Least.	Average.		
61	Tanneries.....	5	590 166	64 500	382 450	1 047	42
62	Forge company.....	1	1 944 666	5 328	213
63	Lead ".....	1	1 656 300	4 543	182
64	Bridge ".....	1	394 166	1 080	43
65	Steam safe company....	1	437 433	1 198	48
66	Glass ".....	1	1 010 000	2 767	111
67	Pipe works.....	1	856 800	2 847	94
68	Pottery.....	1	471 433	1 291	51
69	Bacon works.....	1	75 600	207	8
70	Steamship co. (ocean)...	1	14 908 233	40 844	1 634
71	Carving works.....	1	395 633	1 084	43
72	Wire ".....	1	1 929 566	5 287	211
73	Stables.....	87	5 817 570	23 370	429 133	1 176	46
74	Tube works.....	1	9 169 633	25 122	1 006
75	Nail works.....	1	1 761 200	4 825	193
76	Sewing machine co's....	2	2 215 300	895 100	1 555 200	4 261	170
77	Silver smith.....	1	266 433	730	29
78	Laundry.....	1	1 002 300	2 746	110
79	Bindery.....	1	301 670	826	33
80	Watch factory.....	1	85 000	233	9
81	Fish houses.....	2	959 033	128 700	543 867	1 495	60
82	Chemist.....	1	255 870	701	28
83	Chromos.....	1	434 970	1 192	48
84	Mills.....	34	3 107 700	4 630	852 633	2 336	93
85	Houses and fountains...	2	483 933	262 800	373 367	1 023	41
86	Fountain.....	1	251 100	688	27
87	Ship building.....	1	43 233	118	5
88	House of Correction....	1	12 512 970	34 282	1 371
89	County court house....	1	4 353 830	11 924	477
90	County jail.....	1	1 568 230	4 296	172
91	Police stations.....	10	611 700	169 100	377 540	1 034	41
92	Cemetery.....	1	473 730	1 298	52
93	Organs.....	12	522 933	46 133	184 030	504	20
94	Elevators.....	16	718 430	6 970	150 000	411	16

It is an interesting question, how much water is used for some valuable purpose and how much is simply wasted, and we shall attempt to point out a few places where waste may occur.

First.—There is not as much water pumped or used as is reported, or, in other words, correct methods of calculating the amount consumed are not used.

Where the water supply is pumped from one point to another before being used, it is usual to keep a record of the number of strokes made by the pumps, and from this to calculate the amount pumped. This is a very convenient and safe way, providing the loss of action or the actual amount pumped per stroke is known.* The loss of action in pumps varies in different pumps, and in the Cornish or Worthington, at different times in the same pump.

In a gravitation supply, the methods used are to calculate the quantity passing through the mains, knowing the difference in head between two points, or by shutting off the supply from the reservoir and observing the time required in emptying.

That the correct amounts used are not always given, the following cases are quoted :

In the Boston report for 1863, (page 48), the City Engineer says : "I think it must be apparent that the estimates of the past few years are much too large, and that even the estimate as made for the last year (1863), is greater than it should be by nearly 1 000 000 gallons per day." And again, in 1872 (page 24), he says : "The figures of this table (*i. e.*, of consumption) are but roughly approximate, owing to the want of accurate data for the calculation." In the Cambridge, Mass., report for 1872 (page 6) is said : "A series of observations were made at the engine-house for the purpose of ascertaining the amount of water delivered at each revolution of the pump. As a result, it was found that 300 gallons per revolution was the correct amount, instead of 320 gallons as we had before calculated," a difference of 7 per cent.

The Chicago and Cleveland reports, for 1873 and before, use the theoretical capacity of the pumps in determining the quantity pumped. The loss of action in some of the best engines is from 2.5 to 5 per cent., and in some not so good it is more than 10 per cent., so that when the calculated supply exceeds 95 to 97 per cent. of the capacity of the pumps, it would without explanation seem doubtful.

* Since this was written, a test of one of the Chicago pumping engines shows that the average loss, in fifty hours run, was 15.44 per cent. This, if applied to the table of consumption, would make a considerable difference.

The remedy for this loss would of course be, correct methods of calculation. As this question of supply is of so much importance, it would seem as if some expense could be afforded in order that correct calculations of the amount consumed might be made.

Second.—Water used in condensing steam.

In some cases, water is used from the mains to condense the exhaust steam in creating vacuum, and when this is done, the water must first pass through the pumps and be measured.

There is but one account at hand giving the amount of water used in this way; but the writer has seen a 2 000 000 gallon-engine pumping under 230 feet head, that used water from the force main to condense the steam; the water of condensation was allowed to run back into the pond through an 8 inch-pipe which it filled about half full. The amount used in this way is not known, but it must be quite a per centage of the quantity pumped.

The only report giving the amount of water used in condensing the steam is for Fall River in 1874 (page 63), and we find that for the 92 days of October, November and December, the amount used for condensation was 32 500 000 gallons, or 33.3 per cent. of the total amount pumped during these months. The above amount seems excessive, as it equals 52.8 gallons per revolution.*

Third.—Water lost by leaks in reservoirs.

The reservoir is a point where there is great danger of leaks unless much care was taken during construction. The Boston report for 1857, (page 5, Appendix) says: "Many attempts have been made to stop leaks (in Beacon Hill reservoir), but nothing has as yet been successful." "All structures of stone or brick built to contain water above ground, leak more or less. Beacon Hill reservoir leaks as little as any one in the country." The Boston report for 1867 (page 15) says: "The condition of the East Boston reservoir has been such that it was not deemed safe to raise the water above 15 feet. During the months of September and October, observations were made to determine the amount of leakage. The result showed that with 14 feet of water, the leakage was 19 000 gallons per day, and at 20 feet was 50 000 gallons per day," or 0.1 and 0.4 per cent. of the amount consumed.

Fall River, in 1874, wasted from 50 000 to 100 000 gallons per day over the top of the stand pipes, or 6.6 to 13.2 per cent. Referring to the Woburn report for 1873, (page 4),—"On September 1st, 1873, the engine

* Since this was written, the method of condensing has been changed.

was first put in operation and water forced to the reservoir. It soon became evident that the reservoir would not hold water above 6 feet," and although several thousand dollars were spent in making the reservoir tight, the writer measured a stream, a short time since, that apparently came from the reservoir and it amounted to about 2 per cent. of the total quantity pumped. And the Lynn report for 1873, (page 6) says: "The distributing reservoir mainly built in 1871, was regarded at the date of our last report as almost unserviceable." In 1873, \$41 000 were spent in making the reservoir tight.

Examples of this kind could be added, but the above are sufficient. The reservoirs cannot in all cases be made perfectly tight, but that many leak more than they should is due to want of care during construction, and this can in a great measure be prevented by placing the proper men in charge, those who have had previous experience.

Fourth.—Evaporation in reservoir.

This loss will be the difference between the rain fall and the evaporation. The engineer's report for the additional supply of Boston says, that at Ogdensburg, N. Y., the evaporation was 49.4 inches, at Syracuse 50.2 inches, at Salem 56 inches, and at Cambridge 56 inches; if we assume that 52 inches will be the average evaporation, and 40 inches the rain fall, we shall have a cubic foot of water lost per year for each square foot of area. This for Lowell in 1874 would be 0.6 per cent., Boston, 1873, 0.7 per cent., Philadelphia 0.07 per cent.

Fifth.—Shutting off to repair, and leaks in street mains.

The loss here is probably not large, but must be something. The leaks in the street mains found and repaired are few per mile of pipe, and of course are the largest; but when it is remembered that there are not far from 450 joints per mile of pipe it may not seem strange that an occasional leak should be found, and one of these leaks may occur near a sewer or in gravelly ground, and it may be years before being discovered. A case was recently told the writer where a 4-inch pipe was discovered broken, and had been discharging into a sewer for an indefinite length of time. The estimated discharge was 200 000 gallons per day, or 2 per cent. of the total amount pumped.

The loss by emptying the pipe to make connections is the amount of water in the pipe between the gates. On a 6- or 8-inch pipe, which is the usual size of the street supply mains, gates are not often nearer than 1 000 feet. This would give 1 500 gallons for a 6-inch, and 2 500 gallons for an 8-inch pipe, each time they were emptied.

Sixth.—Poor plumbing.

Penurious landlords or occupants, to save a few dollars will have poor material put into their dwellings, and then suffer leaks which soon occur, to waste water.

This also includes the position of the pipes, for on this, to a large extent, depends that source of great waste, "allowing the water to run to prevent freezing." As a remedy, we would suggest thorough inspection on the part of the officers of the Water Board; not only after the water is let on, but during the time the fixtures are being arranged, and a system whereby plumbers are licensed who must have certain qualifications and make detailed reports of their doings to the Water Board. The Liverpool water engineer says of his experiments: "The chief source of waste has proved to be private pipes and fittings."

The City Engineer of Boston made, July 20th, 1873, some experiments upon the amount of water passing from the Beacon Hill reservoir, between the hours of 1 and 3 o'clock A. M., and found it to be 386 857 gallons (population about 60 000). A party of inspectors were then organized, and the following leaks were stopped: 347 burst service pipes; 491 ball cocks; 1 173 hopper cocks; 1 754 taps; 169 water closets; 50 stop and waste cocks, and 127 hydrants; making 4 111 total. After these leaks had been stopped, observations were again made, October 5th, and the consumption between 1 and 3 o'clock A. M., was 336 294 gallons, a difference of 50 563 gallons, or 606 756 gallons per day. This amount would give 10.1 gallons per capita per day, equal to a saving of 16 per cent. of the average consumption of 1873.

Seventh.—Improper fixtures.

It is not intended to specify all of the improper fixtures that may be in use, but to make one comparison between the hopper and pan water closet.

In a Boston report, we find five cases given, where meters were placed upon hopper closets and allowed to run twelve months; then the pan closets were substituted, and with meter attached, were allowed to run another twelve months. A table of results is given on the next page.

The total amount saved, per year by the substitution, was 3 249 739 gallons, or an average of 249 980 gallons per closet. Boston in 1873, had in use 17 081 hopper closets, and if we assume that each of these wasted the same as the average of the three wasting the least in the above examples, then there are 17.8 per cent of the water wasted by these fixtures.

Number of closets	5	3	1	3	1
Gallons in 12 months, hopper closet....	1 088 750	1 255 470	554 780	494 180	554 800
“ “ “ pan closet.....	384 831	19 839	100 572	113 774	79 205
“ saved in 12 months.....	703 919	1 235 611	454 208	380 406	475 535

The Boston Water Board has been protesting against the use of these closets for years, but it does not seem to have the effect it should.

There were in use in Boston, the following total number of water-closets, and per cent of each kind, as follows :

Year.....	1871.	1872.	1873.
Total	28 733	31 350	34 963
Pan water-closets.....	45.5 per cent.	47.4 per cent.	46.4 per cent.
Hopper water-closets.....	49.1 “	47.5 “	48.9 “
Pull water-closets.....	0.8 “	0.8 “	0.7 “
Self-acting water-closets.....	1. “	0.6 “	0.6 “
Waste water-closets.....	1.5 “	1.8 “	1.7 “
Door water-closets.....	2.1 “	1.9 “	1.7 “

Let us now return to the table and diagram showing the average quantity used per taker per day, and taking Boston as an example, try to discover the reason for the difference in the average amount consumed.

1857. This year the average amount used per taker, per day, was 589.1 gallons, or 10 gallons more than the year previous. Until this year, the Water Board does not seem to have done much, except to *complain* of the great waste. The Water Board thinks the reason the consumption had not increased as before was, that several large consumers had stopped work, and the absence of cold weather.

The subject of meters engrossed a good share of attention, and 24 meters were purchased for the purpose of *trial*.

1858. The consumption was 573.1 gallons per taker per day. The meters bought the previous year “have been tested to a considerable extent and bid fair to be reliable,” and the Board adds 63 more. The Board also came to the conclusion that “the use of meters in several cases appears to be indispensable.”

The Board felt that the ability of the lake to furnish water had been reached, and that unless waste was stopped, some of the larger consumers would have to be shut off.

1859. The average daily consumption per taker was 566.1 gallons. This reduction may be due to several facts, viz. : that during 1858, the lake had fallen, and contained 142 000 000 gallons less than 1857 ; that in the latter part of March, the pipes across Charles river gave way, and the consumption was reduced to 3 500 000 gallons per day, thus leading consumers to think what the result might be if the waste still continued ; that more meters were used, and that the Board "proposed to use them still more extensively."

1860. The average consumption—708.9 gallons per taker—exceeding the consumption of 1859 by 142.8 gallons. The reasons may be—that the dam at the lake was raised in 1859, and during the winter the rain filled the lake ; that the lake was higher in 1859 than in 1858, thus leading consumers to feel there was no necessity to economise as in 1859 ; that the lake having been raised 2 feet, and feeling in 1859 "that the individual consumption had reached its maximum," the Water Board relaxed its vigilance ; and that in the words of the City Engineer, "much of this additional increase of consumption is owing, no doubt, to the increased effective head in the pipes, caused by the new 40 inch-main."

During the early part of this year, several of the leading hotels refused to pay the water rates assessed by the Board through the means of meters. The matter was brought before the Supreme Judicial Court on an injunction to prevent the Water Board from shutting off water from the hotels, and was regarded by both sides as a test case. It was argued September 15th, 1860, but no decision was reached. The amount due from these hotels was \$9 525.60, which at 3 cents per 100 gallons would give 31 752 000 gallons, or 0.5 per cent. of total amount delivered.

1861. "The present Board has adopted energetic measures by the aid of the Police in preventing this evil, (waste), and these measures have been attended with success."

The Court decided the meter case in favor of the city, and the hotels paid the amount due.

More meters had been added during the year, and 0.9 per cent. of the water was metered.

1862. The Board thinks, "this saving or non-use of water is mainly owing no doubt to the number of water meters." The water metered this year was 2.2 per cent.

The diagram shows a sudden falling off in consumption, but this is due to some extent to a change in the manner of estimating the quantity consumed. The City Engineer says : "had the methods employed for a

few years past, been used this year, the daily average would have been 18 625 000 gallons, instead of 16 238 500 gallons."

1863. A circular was issued during the year which seemed to have good effect.

1864. Less water was used, "and it is undoubtedly owing to increased vigilance and care on the part of our citizens, inspired by fear of a short supply, and by extra exertions of the Board and its officers in tracing out sources of waste."

1865 and 1866. A system of inspection was adopted, the number of meters increased, and amount of water metered also increased being 8.8 per cent.

1867. The number of meters were increased in about the same per cent. as the increase in takers, but the total amount metered was only 8.2 per cent., or as compared with total amount consumed, 6.8 per cent. less than 1866.

1868. The number of meters was increased; the quantity of water metered was increased and the consumption was slightly increased, and the number of consumers increased. It is possible that the increase in amount consumed may be accounted for on the supposition that consumers whose water is measured, do not realize the quantity they are using until the bills come in, and it is in the second year that the reduction is felt, as per 1869.

1870. Although the number of meters was less, the amount of water metered was increased, so that there was a reduction in amount consumed.

1871. An increase in the meters used, and per cent of water metered, and decrease in amount consumed per taker.

1872. Decrease in meters used, increase in water metered and in amount consumed.

1873. Increase in meters, decrease in water metered, and increase in amount consumed.

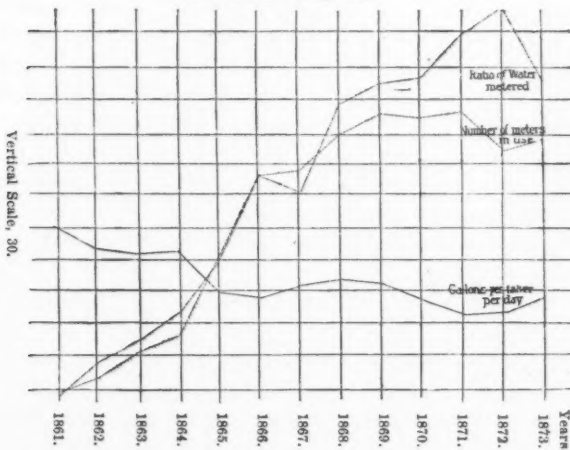
To show this plainer we give the table and diagram on the next page. The table begins in 1861, for this is the year in which meters came into practical use; the Supreme Judicial Court having decided that the consumer must pay by meter when required by the Water Board.

From 1861 to 1865 inclusive, the rate by meter varied from 2 to 6 cents per 100 gallons, according to the amount used (or the water was sold by "wholesale or retail.") There is one objection to the wholesale rate of selling water, in that a consumer by wasting a few hundred

NUMBER OF METRES, RATIO OF WATER METERED, &C.

Year.	Number of meters in use.	Ratio of water metered to total amount consumed.	Average number of gallons per taker per day.	Remarks.
1861	104	0.009	713.7	Rates assumed to be 3 cents per 100 gallons, the actual, varying from 2 to 6 cents.
1862	161	0.022	631.4	
1863	254	0.03	610.9	
1864	312	0.039	616.8	
1865	586	0.057	460.6	November 22d, this year, meter rates were made uniform at 3 cents per 100 gallons.
1866	879	0.088	440.6	
1867	895	0.082	482.7	
1868	1 021	0.113	495.9	
1869	1 089	0.12	478.4	Metered water given for 9 months, one-third added for 12 months.
1870	1 076	0.122	415.3	
1871	1 091	0.137	360.2	
1872	955	0.147	370.2	
1873	977	0.12	421.5	

DIAGRAM. (Fig. 13).



gallons per day will have the water at a less rate, and *less total cost*. Since 1865, the rate has been uniform, at 3 cents per 100 gallons.

The rate of 3 cents per 100 gallons has been used in the above table in reducing the amount of money received, to gallon metered, as a fair

average for 1861 to 1865, and exact since 1865. The reports for 1868 to 1872 inclusive, give the quantity metered for nine months only; therefore, for comparison for the year, one-third has been added.

It will be seen that when the *least* number of meters was used, and the *least* quantity of water was metered, the *consumption per taker* was the *highest*, viz.: 713.7 gallons. Per contra, it will also be seen that when the number of meters was *greatest* and (with one exception) the quantity of water metered was the *highest*, the *consumption* was the *lowest*. As a rule, when the number of meters in use and the ratio of water metered to total quantity consumed is increased, the quantity consumed per taker is decreased, and the conclusion we come to is, if a permanent reduction of waste is desired, meters must be used; all other means being but temporary.*

There are other points to which attention might be drawn, but this paper has grown to a greater extent than was intended. If it serves to bring out, or gives to others, desired information, its object will be attained.

* See remarks of J. Herbert Shedd, Vol. V, page 254. Providence, R. I., with a population of about 91 000, ratio of 0.166 takers to population, and 798 inhabitants per mile of pipe, uses 22.5 gallons per capita.

ERRATA.—On page 417, (Volume V), twenty-sixth line, for “Segnoia” read “Sequoia”; on page 71, (current volume), seventh line, for “formations” read “foundations”, and thirty-third line, for “length” read “height”; on page 97, thirty-third line, for “or” read “as a”; on page 101, thirty-third line, for “for” read “from”; on page 103, twenty-seventh line, for “6.17 693 65” read “6.17 693 66”; on page 104, twenty-eighth line, for “are” read “should have been”; on page 105, fourth line, for “7.258 390 6” read “7.259 374 0”, and on page 106, third line, for “7.081 937 6” read “7.083 514 8”.

Also on Plate X, Fig. 8, for “222 300” read “232 300”; for “171 800” read “161 800”, and for “144 700” read “134 700”.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CLX.

APPROXIMATE

DETERMINATION OF STRESSES IN THE EYE-BAR HEAD.

A Paper by WILLIAM H. BURR, C. E., Junior of the Society.

PRESENTED FEBRUARY 17TH, 1875.

PRELIMINARY.—As the title indicates, this discussion is nothing more than an approximation; the extreme intricacy of the problem if treated exactly, and the unknown value of one of the constants, scarcely allow of anything else. The essential agreement, however, of the results with those of experiments show that the approximation is a close one. Two or three assumptions are made that are not exactly borne out by the facts of the case, but the deviations from the truth are shown to be so small that they are not of serious importance.

The case of the perfectly fitting pin is treated, not because such a one exists, but many of the formulæ found in Case I are used in Case II, and the subject would not be complete without it.

Allowance for friction is not involved in the examples worked out. The extent of that influence is exceedingly uncertain, and the omission does not favor the eye-bar head, but is against it.

It is to be very much regretted that the final formulæ are so tediously long, and of such a high order. Simple and approximate equations are not easy to obtain from the correct ones, but the "rule of thumb" given for laying out a head is not far wrong for ordinary relations between the diameter of pin and width of bar.

The object of an equation, it is to be remembered, is not altogether to supply a method of deducing the actual value of the unknown quantity which enters it, although that may be the most important part.

If the points of maxima and minima, or those of other relative values are indicated, an equation is not without utility.

In that part relating to the imperfectly fitting pin, a play of 0.02 inches is assumed between the pin and its hole. This is not an extreme case, but is about the best practice.

The tensile and compression stresses in the head in front of the pin are considered to exist independently of each other. This of course, is not true, but to make the proper allowances presupposes a closer degree of knowledge of the molecular nature of material than has yet been attained. Within the limit of elasticity however, this probably leads to no essential error.

CASE I.—For the first case, let it be assumed that the pin accurately fits the pin-hole. In the following investigation of this case, the effect of friction between the pin and bar will be disregarded, and the pressure between the two surfaces will consequently be in the direction of the common normal passing through the centre of the pin-hole. The pressure in the direction of the axis of the bar on the diameter of the pin normal to that direction will be uniformly distributed over that diameter, and the intensity of the pressure, *i. e.*, pressure per unit of

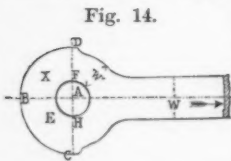


Fig. 14.

area, will be the same as that existing uniformly around the semi-circumference of contact between the pin and pin-hole. If the applied force is exerted in the direction of the arrow, Fig. 14, there will only be a semi-circumference of contact between the pin and bar.

These statements will be evident after a little consideration. No bending takes place at *B*, or any other point in front of the line *DC*; consequently the case is the same as that of a flexible cord around a cylinder without friction, in which the intensity of the pressure between the surfaces of contact is constant, and in the direction of the common normal.

Now, since this last condition exists, the laws governing the intensities of the stresses in the metal *BCED*, in front of the pin, are the same as those for fluid pressure in a closed cylinder; therefore, the intensity of the pressure on the diameter *FH*, in the direction of the axis of the bar, is the same with that on the semi-circumference *HEF*, and the product of this intensity by the diameter of the pin-hole is equal to the total tension to which the bar is subjected.

Let r_0 = radius of pin or pin-hole = $AH = SF$, r' = any radius between r_0 and R , R = exterior radius of head = $AB = AD$, t =

thickness of bar, t_1 = thickness of head, w = width of bar, f = hoop tension at distance r' from centre and f_0 that at inside of head, q = radial pressure at distance r' from centre, and q_0 at inside of head, and f_1 tension per square inch in the bar. Also put $P = f_1 w t$ = total tension in bar. From what has been said before, there evidently results:—

$$q_0 t_1 r_0 = \frac{1}{2} P = \frac{1}{2} f_1 w t \therefore q_0 = \frac{f_1 w t}{2 r_0 t_1} \dots\dots\dots (1)$$

In Rankine's Applied Mechanics, Article 273, it is shown (putting q_1 in that article equal to zero, since it is inappreciable in this case when compared with q_0) that:—

$$f = \frac{q_0}{R^2 - r_0^2} \left\{ \frac{R^2 + r'^2}{r'^2} \right\} \dots\dots\dots (2)$$

$$q = \frac{q_0}{R^2 - r_0^2} \left\{ \frac{R^2 - r'^2}{r'^2} \right\} \dots\dots\dots (3)$$

It is well to notice in Eq. 1, if $w = 2 r$ and $t = t_1$ that $q_0 = f_1$.

Calling, with Rankine, the tension around the circumference "hoop tension" and making $r' = r_0$ in Eq. 2, the maximum intensity of hoop tension is obtained:—

$$f_0 = q_0 \left\{ \frac{R^2 + r_0^2}{R^2 - r_0^2} \right\} \dots\dots\dots (4)$$

and radial pressure at some point:—

$$q_0 = q_0 \dots\dots\dots (5)$$

The minimum intensity of hoop tension at distance R is :

$$f_R = \frac{2 q_0 r_0^2}{R^2 - r_0^2} \dots\dots\dots (6)$$

Intensity of radial pressure at same point:—

$$q_R = 0 \dots\dots\dots (7)$$

If R is the quantity sought for in terms of r_0 , then results from Eq. 4.

$$R = r_0 \sqrt{\frac{f_0 + q_0}{f_0 - q_0}} \dots\dots\dots (8)$$

If the thickness of the head differs from that of the bar, f_0 and q_0 will depend on the ratio of the two quantities t and t_1 .

If R is given some value in terms of r_0 , t_1 can be determined by taking a proper value for f_0 . Substituting in Eq. 8, q_0 from Eq. 1:—

$$R = r_0 \sqrt{\frac{f_0 + \frac{f_1 w t}{2 r_0 t_1}}{f_0 - \frac{f_1 w t}{2 r_0 t_1}}} \dots\dots\dots (9)$$

Suppose $R = 2 r_0$ in Eq. 9, after reducing:—

$$t_1 = \frac{5 f_1 w}{6 f_0 r_0} t \dots \dots \dots (10)$$

Let $f_1 = f_0$, and $r_0 = \frac{1}{2} w$ then:—

$$t_1 = 1 \frac{2}{3} t \dots \dots \dots (11)$$

Therefore, the head should be thickened to the extent shown by Eq. 11, in order that the maximum intensity of the tensile stress in the eye-bar head may not be greater than the intensity of the same in the bar Eq. 4 gives:—

$$q_0 = \frac{3}{5} f_0 \dots \dots \dots (12)$$

Suppose that $t = t_1$ and $f_1 = f_0$ in Eq. 10:—

$$r_0 = \frac{5}{6} w \dots \dots \dots (13)$$

Eq. 13 gives the radius of the pin when the maximum intensity in the head equals that in the bar, the thickness of the head remaining the same with that of the bar. Eq. 4 again gives:—

$$q_0 = \frac{3}{5} f_0 \dots \dots \dots (14)$$

since $\frac{R}{r_0}$ is not changed.

From what precedes, it is seen that the eye bar head, perfectly formed takes the shape given it in Fig. 14. If the head is not very much thickened its radius will suddenly decrease after passing back of diameter DC .

Again in Eq. 9, put $R = 3 r_0$, then:—

$$t_1 = \frac{5 f_1 w t}{8 r_0 f_0} \dots \dots \dots (15)$$

Let $f_1 = f_0$ and $r_0 = \frac{1}{2} w$:—

$$t_1 = 1 \frac{1}{4} t \dots \dots \dots (16)$$

In Eq. 15 suppose $t = t_1$ and $f_1 = f_0$:—

$$r_0 = \frac{5}{8} w \dots \dots \dots (17)$$

Eq. 4, also gives: $q_0 = \frac{4}{3} f_0$.

Before the pin can be torn through the metal in front of the eye,

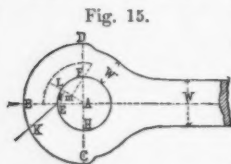


Fig. 15.

either of the halves, $BEFD$ or $BEHC$, Fig. 15, must be ruptured transversely at DF or HC , where the metal is suddenly brought down from the width required in front of the eye. Let w' equal the width shown in Fig. 15, and f' the greatest tensile intensity existing at F . The

width w' will be taken as if lying on DC , and it will be supposed that rupture has taken place at BE . Moment of resistance of section w' will be :

$$M = f' \cdot \frac{t w_1^2}{6} \dots \dots \dots (18)$$

In this instance, as before, q_0 will represent the constant intensity of normal pressure around HEF . The lever arm of any small part of q_0 acting at any point L , Fig 18, is $(r_0 + \frac{1}{2} w') \cos. \Theta$; Θ being measured from AB . This small part of q_0 is $q_0 r_0 d \Theta$, and its moment :—

$$d(m) = q_0 r_0 (r_0 + \frac{1}{2} w') \cos. \Theta d \Theta.$$

This moment is taken around the centre of w' .

$$\therefore m = \int_0^{\frac{\pi}{2}} d(m) = q_0 r_0 (r_0 + \frac{1}{2} w') \dots \dots \dots (19)$$

The total moment required to break off both halves (at H and F') will of course be double that shown by Eq. 19. At the instant of rupture Eq. 19 = Eq. 18, if f' represents the ultimate resistance of the material ;

$$\therefore q_0 r_0 (r_0 + \frac{1}{2} w') = f' \cdot \frac{t w_1^2}{6} \dots \dots \dots (20)$$

$$\therefore 2 q_0 r_0 = f' \cdot \frac{t w_1^2}{3 (r_0 + \frac{1}{2} w')} \dots \dots \dots (21)$$

Eq. 21 gives the amount of tension in the bar which will cause the two transverse ruptures at H and F' when connection between B and E is broken.

Comparing Eq. 21 with Eq. 1, it will be seen that if $r_0 > \frac{1}{2} w'$, and $f' f = P$ must always be much greater than than $2 q_0 r_0$. Therefore, nothing short of extravagant amounts of metals at F and H can add to the strength of the eye by transverse resistance at DF and HC . Since the intensity of the ultimate crushing resistance of wrought iron in short pieces can be taken at about four-fifths the intensity of its tensile strength, it is seen from Eq. 16 and that immediately following Eq. 17, that the radius of the pin should be about one-half the width of the bar, and the thickness of the head, one and a quarter that of the bar.

No account has been taken of the compressibility of the material since the section of the pin will remain circular, because the intensity of the normal pressure exerted on its surface is constant. The exceedingly slight change in the value of the radius will cause no appreciable variation in the results given.

Eq. 2 shows that the intensity of the hoop tension decreases quite rapidly as the distance from the centre of the eye increases when the

thickness of the head is supposed to be constant. Now it is evident, if the thickness of the head is variable and decreases according to the law governing the varying intensity of the hoop tension, that the quantity of material in the head will be the least possible.

Agreeable to the condition under which Eqs. 15 and 16 were derived, let f in Eq. 2 be displaced by $y f_0$; y being some variable so chosen that when r' is equal to r_0 , y shall equal 1.25.

Making this substitution and putting $\frac{4}{5} f_0$ for q_0 in Eq. 2, there results :

$$y = \frac{4 r_0^2}{5 (R^2 - r_0^2)} \left\{ \frac{R^2}{r'^2} + 1 \right\} \dots \dots \dots (a)$$

Put $r' = r_0$ and 1.25 for y :—

$$1.25 = \frac{4 (R^2 + r_0^2)}{5 (R^2 - r_0^2)} \dots \dots \dots (b)$$

Dividing Eq. (a) by Eq. (b) :

$$y = \frac{5 r_0^2}{4 (R^2 + r_0^2)} \left\{ \frac{R^2 + r'^2}{r'^2} \right\} \dots \dots \dots (c)$$

This equation gives the thickness for any value of r' under the conditions assumed.

In Eq. (a) let y be the ordinate of the curve and r' (displaced by x) the abscissa, and write for convenience :

$$y = A \left(\frac{B}{x^2} + 1 \right) \dots \dots \dots (d)$$

Since Eq. (d) is of the third degree, it is not an equation to any of the conic sections. Differentiating :

$$\frac{dy}{dx} = -2 AB \frac{1}{x^2 dx^2} = \frac{6 AB}{x^3};$$

therefore, the curve is convex towards the axis of abscissa, and has no point of contrary flexure. Eq. (d) shows that the curve has an asymptote parallel to the axis of x , and at the distance A from it, consequently the curve itself never touches the axis.

The area of a section whose plane passes through the origin; *i.e.*, the centre of the eye, will be found equal to that of half the body of the bar, as it should.

The curve, of course, $= \int_{r_0}^R y dx$; making substitution from above :

$$\int_{r_0}^R y dx = \frac{AB}{r_0} - \frac{AB}{R} + (R - r_0) A.$$

Making the substitution in terms of W assumed in Eqs. 15 and 16, the area will widen to $\frac{W}{2}$ since the thickness of the bar was taken at unity.

In Eq. (c) let $r_0 = 2$ and $R = 6$ inches, then by giving r' values shown below, the ordinates opposite will be found.

$r' = 2$ inches.	$y = 1.25$ inches.	$r' = 5$ inches.	$y = 0.305$ inches.
$r' = 3$ "	$y = 0.625$ "	$r' = 6$ "	$y = 0.25$ "
$r' = 4$ "	$y = 0.406$ "		

Fig. 16.

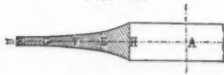


Fig. 16 is a section on the line EB , Fig. 15, with the ordinates calculated above; A , Fig. 16, being the center of the eye, and r' is measured from there.

The only shearing stress which exists in front of the pin, in the material of the head, is that resulting from the two stresses q and f , and the formulæ required are found in Rankine's "Applied Mechanics," Art. 112.

Let the intensity of this shearing stress $= S$. Take the circumstances the same as those under which Eq. 14 was found. Eq. 5 of the article named is the one to be used, and it is applied to any point of the semi-circle of contact. $p_x = f_0$, and $-p_y = \frac{3}{5}f_0$; hence:—

$$\frac{p_y - p}{2} = S = 0.8 f_0 \dots\dots\dots (e)$$

Again in Eq. 15 make: $t_1 = t$; then $f_0 = \frac{5 f_1 w}{8 r_0}$; also let r_0 equal $\frac{w}{2}$; then $f_0 = \frac{4}{5} f_1 = p_x$; hence Eq. 5 gives $q_0 = \frac{4}{5} f_0 = f_1 = -p_y$;
 $\therefore S = 1.125 f_0 \dots\dots\dots (g)$

Eq. (e) and (g) give the maximum value of S in each case. Other values possess no particular interest, but may easily be found.

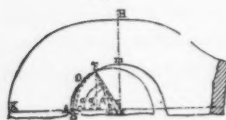
CASE II.—In the previous case the semi-circumference of the pin is supposed to touch at all points the interior surface of the link head when no stress exists in the bar. This circumstance never exists, however, in practice, but the diameter of the pin is from $\frac{1}{32}$ to $\frac{1}{100}$ inch smaller than that of the pin-hole.

If the material were rigid, contact would exist along a line simply, but the material is elastic, and its elasticity allows a surface of contact to be formed between the pin and pin-hole. The extent of this surface will depend on the modulus of elasticity of the material, the intensity of stress in the bar and the depth of metal in front of the pin.

In Fig. 16 suppose that the line AF lies in the axis of the bar, then evidently a half of the arc of contact BE will lie on each side of this line, and at equal distances from it; the intensity of the pressure on the pin will be the same. The effect of friction, if such exists, will be

neglected, and the direction of the pressure between the surfaces of contact will be that of a normal to the two surfaces. In this investigation two assumptions will be made, neither of which involves an appreciable error.

Fig. 17.



The intensity of the normal pressure spoken of above is assumed to vary directly as its angular distance on the pin from the point *E*, Fig. 17. A little but careful consideration of the question will make it appear that this is almost, if not exactly true. The distance *AB*, the compression of the pin, is so small that if *ABE* be developed on *AE*, any one of the elements, as *AB*, drawn perpendicular to *AE*, will be essentially perpendicular to *BE* and a direct function of its distance from *E*. The real law, of course, would be shown by drawing elements truly perpendicular to *BE*; these representing the intensities of the stress at the points from which they are drawn, will also be direct functions of their distances from *E*, measured on *BE*.

The second assumption is that the curve of contact, *BE*, is a circle. It is supposed that circular pins are used. Recent experiments on the strength of the heads of eye bars have shown that when a proper size is given to the pin, *i. e.*, a size capable of bringing out the maximum resistance of the bar, no radial element of the pin is strained beyond the elastic limit; also, remembering that the pin very nearly fits the hole, it will be seen that the curve *BE*, Fig. 17, is essentially a circle whose radius lies between those of the pin and hole.

AB evidently represents the greatest compression among the radial elements of the pin and *BC* the same thing for the eye-bar head. Although these, *as lines*, are not equal, yet they represent equal intensities of stress. The same remark may be made in reference to any other point of contact, since action and reaction must be equal at all points.

As *AF* is a line of symmetry for the normal forces acting on the surface of contact, it will only be necessary to consider those on one side of it; let the forces which act on the same side with *E* be discussed. Although the intensity of the normal stress between the surfaces may vary at all points of the curve *BE*, for a differential of that line it may be considered constant. Having once determined the intensity of this stress at any point, together with the radius of the curve of contact, it will hereafter be shown that the value of the tension around the pin may be obtained by means of some of the formulæ of Case I. The part

of the link head KBE , Fig. 17, is held in equilibrium—supposing KB to be fixed—by the tension applied to the bar transmitted through the section MH , and the pressure of the pin on the surface projected in the line BE . Suppose a section, whose plane cuts the curve of contact and passes through its centre, to be taken at any point between B and E , and let all the normal stress between that section and the point E , exerted by the pin against the head, be resolved into two components, the one acting normal to the section and the other in the direction of the axis of the bar; the former will be called the normal and the latter the axial component of the pressure. Similar to the case of stresses in the material of a closed cylinder, since the pressure on the surface of contact is normal to it, the principal axis of stress in the metal in front of the pin will be in the direction of a radius to the curve BE , and parallel to it (the curve). Now, since the only force which is supposed (for the time being only), to induce internal stress in the mass KBE is the normal pressure on BE , the internal stress perpendicular to any section taken as stated before, must be equal to the “normal component of the pressure” acting on that section. It will first only be necessary to find this normal component for the section KB ; its general value, however, will be obtained in the first part of the investigation.

A bending stress also exists at all points except one, between K and H , and must be taken into account. This stress is compressive on the interior surface of the eye-bar head between B and some point O , and tensile from O to M . The opposite conditions hold on the exterior surface. The change in the nature of the stress at O is accounted for by the fact, that a point of contra-flexure exists in the radial section whose plane passes through O . The value of the moment about the centre of any section, from which results this bending stress, is the difference of the moments of the tension in the section and MH the normal pressure against the surface of contact, acting with their proper lever arms.

Since the “axial component of the pressure” and the tension in MH do not act parallel to any section under discussion, with the exception of KB , their difference, as they act in opposite directions, will have a component normal to the section and distributed uniformly over it. Consequently the intensity of this stress in any section will be constant.

It will now be evident that for every radial section in front of the pin, except KB , there will be three sources of tensile stress parallel to the curve of contact on the exterior surface of the head, and

on the interior surface, two sources of tensile and one of compressive stress. By estimating these for a number of sections and combining them in a proper manner, the correct shape and dimensions of an eye-bar head may be determined.

Let the intensity of the normal pressure between pin and hole at *B*, Fig. 17, = q_0 , the general expression for that intensity = q , the radius of the pin = r , the radius of the pin hole = r_1 , the distance from the centre to the outside of the head (variable) = R , and the angle of contact (measured from centre of pin) BGE , = α .

In Fig. 18, suppose that the circle *ADE* refers to the circle of contact.

Fig. 18.

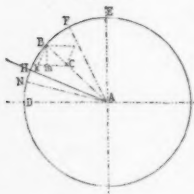


Fig. 17 shows the location of that circle. Then we have $AE=r_2$; $FAE=\gamma$; $FAH=\Theta$; $FAD=\alpha_2$; $FAB=\Theta'$; $FAN=\beta$, and $NAD=\delta$.

The angle Θ' is the general value of any angle less than Θ , measured from AF . The "normal component of the pressure" will first be found in its most general form, from which it is easy to pass to that for any special case. Since such a case will finally occur, the intensity of the pressure between the pin and head is considered constant for some distance DN on each side of the line of symmetry AD ; consequently: $FAD - FAN = \alpha_2 - \beta =$ the angular distance over which the intensity q_0 is constant. The angle $FAN = \beta$, is the angular distance over which q varies.

Subscript (2) belongs to data referring to the circle of contact, which Fig. 18 is supposed to show. Draw the diagram of forces as shown at BKC , and suppose that DA lies in the axis of the bar, then:

$$BC = r_2 q d \Theta'$$

Let N stand for the "normal component" sought, then BK being perpendicular to AH :

$$BK = d(N) = r_2 \operatorname{cosec}. (\Theta + \gamma) q \cos. (\Theta' + \gamma) d\Theta'$$

Now, the intensity of pressure between pin and head varies as the angular distance from E toward N ;

$$\therefore q = \frac{\Theta'}{\beta} q_0 \dots\dots\dots (22)$$

Remembering that the differential of N must consist of two parts, for the constant and varying value of q :-

$$N = r_2 q_0 \operatorname{cosec}. (\Theta + \gamma) \left\{ \int_{\beta}^{\Theta} \cos. (\Theta' + \gamma) d\Theta' + \frac{1}{\beta} \int_0^{\beta} \Theta' \cos. (\Theta' + \gamma) d\Theta' \right\} \dots\dots (23)$$

$$\therefore N = r q_0 \operatorname{cosec} . (\Theta + \gamma) \left\{ \sin . (\Theta + \gamma) + \frac{\cos . (\beta + \gamma) - \cos . \gamma}{\beta} \right\} \dots (24)$$

The section taken in the preceding equations is on *FAH*, Fig. 18, at an angular distance Θ , (supposed to be greater than β from *FA*).

If Θ were less than β (as is shown), the first part of the integral expression for N would be omitted, then :—

$$N = r_2 q_0 \frac{\operatorname{cosec} . (\Theta + \gamma)}{\beta} \left\{ \Theta \sin . (\Theta + \gamma) + \cos . (\Theta + \gamma) - \cos . \gamma \right\} \dots (25)$$

If q varies throughout the whole extent of the angular distance α_2 , then $\beta = \alpha_2$, and $(\alpha_2 + \gamma) = 90^\circ$ in Eq. 25.

Therefore, for section *KB*, Fig. 17, as $\Theta = \alpha_2$:—

$$N = r_2 q_0 \frac{\alpha_2 - \sin . \alpha_2}{\alpha_2} = r_2 A q_0 \dots \dots \dots (26)$$

It is seen that all the Eqs. 24-26 may be written :— $N = r_2 A q_0$.

It will be necessary next to find the amount of contact between the pin and interior surface of the link head, and in order to do that the maximum radial compressions *AB* and *BC*, Fig. 17, must be known. The radial compression *BC* of the head is, of-course, the sum of all the compressions of the concentric layers which make up *KB*, and the law of the variation of the radial pressure q between *B* and *K* supplies the method of obtaining the sum. The general conditions to which this varying radial pressure is subject are that it shall equal q_0 at *B* and zero at *K*; also that the total tensile stress co-existent in the section *KB* with q , shall equal the "normal component of the pressure," as given by one of Eqs. 24-26.

The first equation deduced will be for that case in which the normal pressure between the pin and head varies uniformly from *B* around to *E*: hence Eq. 26 is the one to be taken for the value of N .

Let t stand for the intensity of the tensile stress at any point in the section *BK*, at a distance R , from the centre of the pin-hole, then the equation which is to be fulfilled is :—

$$\int_{r_1}^R t dr = N = r_1 A q_0 \dots \dots \dots (27)$$

Eq. 27 will be at once recognized as one identical with that which holds for a closed cylinder, the intensity of the pressure against whose interior surface is q_0 , and whose radius is $r_1 A$. The conditions of q , being equal to q_0 at *B*, and zero at *K*, Fig. 17, are introduced in all the equations of Case I, referring to radial and circumferential stresses. Put r'_1 for $(r_1 A)$, and wherever R occurs put R' , also let q be the intensity of the

radial stress in the head at the distance r' from the centre of the pin-hole.

From Eq. 3, Case I, for an interior radius r'_1 , and exterior R :

$$q = \frac{q_0 r'^2}{R^2 - r'^2} \left(\frac{R^2 - r'^2}{r'^2} \right) \dots\dots\dots (28)$$

If E stands for the modulus of elasticity of the material, then evidently,

$$BC = \int_{r'_1}^R \frac{q}{E} dr' \dots\dots\dots (29)$$

substituting for q and integrating:

$$BC = \frac{q_0}{E} \frac{r'^2}{R^2 - r'^2} \left(\frac{R^2}{r'_1} + r'_1 - 2R \right) \dots\dots\dots (30)$$

r'_1 and R in Eq. 30 do not really refer to the centre of the pin-hole of the link-head, under discussion, and they must be replaced by values in terms of R and r_1 by means of the equation:

$$r'_1 = r_1 A \dots\dots\dots (31)$$

Since A can never be greater than unity, there results:

$$R = R - (1 - A) r_1 \dots\dots\dots (32)$$

substituting from Eqs. 31 and 32 in Eq. 30, and reducing:—

$$BC = \frac{q_0}{E} \frac{1}{\frac{1}{Ar_1} + \frac{1}{(R - r_1)}} \dots\dots\dots (33)$$

AB is the maximum compression of any of the elements of the pin, and since the radial pressure, q_0 , in the pin does not vary along the radius* :—

$$AB = \frac{q_0}{E} r \dots\dots\dots (34)$$

In order to find α , the angle of contact measured from the centre of the pin, it will be seen, by reference to Fig. 17, if it is supposed that the pin occupies its position under the influence of applied forces, that:

$$FG = r_1 \cos. \alpha_1 - r \cos. \alpha = r_1 - r + AB + BC \dots\dots (35)$$

Subscript (1) belongs to data referring to the centre of the pin-hole. Substituting for $\cos. \alpha_1$ in the first value of FG :—

$$(r_1^2 - r^2 \sin.^2 \alpha)^{\frac{1}{2}} - r \cos. \alpha = r_1 - r + AB + BC \dots\dots (36)$$

Putting second member of Eq. 36 equal to K , transposing $r \cos. \alpha$, squaring and reducing :—

$$\cos. \alpha = \frac{r_1^2 - r^2 - K^2}{2rK} \dots\dots\dots (37)$$

Substituting in the value of K those of AB and BC from Eqs. 33 and 34, then substituting in Eq. 37 :—

* This is not the exact truth, but sufficiently near for the purpose.

$$\cos. \alpha \left\{ 2 r r_1 - 2 r^2 + 2 r \frac{q_0}{E} \left\{ r + \frac{1}{Ar_1} + \frac{2}{R-r_1} \right\} \right\} =$$

$$2 r r_1 - 2 r^2 - \frac{q_0 2 (r_1 - r)}{E} \left\{ r + \frac{1}{Ar_1} + \frac{2}{R-r_1} \right\} -$$

$$\frac{q_0^2}{E^2} \left\{ r + \frac{1}{Ar_1} + \frac{2}{R-r_1} \right\}^2 \quad (38)$$

As done before, let Θ stand from the angle measured from the line GE towards GK , G being the centre of the pin. Also, let q stand for the intensity of normal pressure at the point of contact, denoted by Θ . Then, since q varies directly as Θ :

$$\frac{dq}{d\Theta} = \frac{q_0}{\alpha} \therefore q = \int_0^{\Theta} \frac{q_0}{\alpha} d\Theta = q_0 \frac{\Theta}{\alpha}$$

which value was immediately written before.

Put P for the total amount of tension impressed on the bar. To get an expression for it in terms of α , put r' for the perpendicular distance from AF to any point whose angular position is located by Θ from GE . Let $\gamma = 90 - \alpha$, then $r' = r \cos. (\Theta + \gamma)$. Therefore:—

$$\frac{1}{2} P = \int_{\alpha}^0 q dr' = \frac{q_0 r}{\alpha} (1 - \cos. \alpha) \dots \dots \dots (39)$$

The upper limit of the integral is taken at zero, because $\frac{1}{2} P$ (considered as a variable) increases as the angle Θ diminishes.

By reference to Eq. 38, it is seen that the angle α depends on R , as it should. As R increases, the diameter of the pin and tension in the bar remaining the same, α will increase; and as the cosine is an inverse function of the arc, $\cos. \alpha$ is an inverse function of R .

When Eq. 38 is used in seeking α , the last term of the second member, involving the square of the reciprocal of the modulus of elasticity, may be omitted without appreciable error.

Eqs. 38 and 39 contain the unknown quantities, q_0 , α and R , and any two of them may be deduced if the third is given. The solution for α , however, either of the two others being given, would be so intricate that the equation would have little real value.

If by any means, α should be known, or if a bar already in existence is under discussion, the formulæ are of easy application; for, in the first place, q_0 and R could be directly obtained; and, in the second, only a comparatively simple case of approximation would have to be resorted

to. This approximation, when P and R are known, is best performed by assuming the most probable value of α and calculating q_0 from both Eqs. 38 and 39; when these two values agree, α has been correctly taken and the problem is solved. After the intensity of the pressure normal to the surface of contact is found, Eqs. 2 and 3 of Case I may be used in finding the radial pressure and circumferential stress, or hoop tension, at any point of the head by replacing, in those formulæ, r_1 by Ar_1 and R by $R - (1 - A) r_1$, then remembering that r' is to be replaced by $r' - (1 - A) r_1$ and estimated from the centre of the actual circle of which r_1 is the radius. Again, suppose that R and q_0 are given, and that it is desired to strengthen the head by thickening, if R does not give sufficient depth of material in front of the pin. The degree of accuracy with which the pin fits the hole will be given by $(r_1 - r)$. Making the substitution in Eq. 38, α will have to be determined by a simple approximation; substituting this in turn, in Eq. 39, P will be known, and, consequently, the intensity of the tension in the bar. If t^1 is the calculated intensity, and t the desired; then $\frac{t}{t^1}$ is equal to the thickness of the head divided by that of the bar, and the thickness of the latter multiplied by this ratio gives that of the head. From what has preceded, together with an examination of Eq. 38, it will be evident that, except in the improbable case of α being given, it will be almost necessary to approximate in getting a value of that angle.

It is hardly necessary to say that the preceding formulæ are deduced for a bar whose thickness is unity, but are equally applicable to one of any thickness whatever.

The radius of the circle of contact may now easily be found. In Fig. 17 let x be the position of the centre of that circle. Denote by r_2 the radius sought, and by α_2 the arc of contact. Then there results:

$$r_2 \cos. \alpha_2 - r \cos. \alpha = r_2 - r + (BA) \dots \dots \dots (40)$$

$$\text{Also, } \sin. \alpha_2 = \frac{r}{r_2} \sin. \alpha.$$

Substituting this value of $\sin. \alpha_2$ in the trigonometrical formulæ,

$$\begin{aligned} r_2 \cos. \alpha_2 &= \sqrt{1 - \sin.^2 \alpha_2} \text{ and } - \\ \cos. \alpha_2 &= \frac{\sqrt{r_2^2 - r^2 \sin.^2 \alpha}}{r_2} \dots \dots \dots (41) \end{aligned}$$

Putting the value of $\cos. \alpha_2$ in Eq. 40, then squaring and reducing:

$$r_2 = \frac{r^2 \sin.^2 \alpha + H^2}{2H} \dots \dots \dots (42)$$

contra-flexure will be tensile in the material at the interior surface of the link head, and compressive on the exterior.

It has been deduced from experiments that if DE , Fig. 19, is about two-thirds W , e will not be less than three-tenths W . In these experiments, the pins were about seven-eighths the width of the bar in diameter.

For different proportions among the data, a value of e will have to be selected by the judgment, since it is essentially impossible to calculate for it with any degree of probability of an accurate result. Fortunately this quantity so enters the formula that any small mistake in its value does not lead to an error of any importance in the result.

The values of both the moments, *i. e.*, for the bar tension and outward pressure, will first be found for the general case by supposing any section under consideration, and that the intensity of the pressure of contact is constant for some distance on either side of the axis of the bar. Fig. 18 will be referred to, with the corresponding symbols. In that figure, $NAD = \delta$ is the angular distance over which q_0 is constant, and $\beta = FAN$, the distance over which q varies, while Θ measured from AF , locates the section to be considered. The outward moment will be composed of two parts when the section around whose centre the moment is taken is at angular distance Θ , from AF , of any value between β and α_2 , on account of the constant and varying intensities of pressure.

When Θ is less than β , the moment will have but one part, since q is not then constant at any place.

Suppose the section AB , Fig. 19, to be under consideration, and let it correspond to AB , Fig. 18, then the outward moment of the normal pressure will be about the point B , Fig. 19, in the middle of the section. As before, put q for the general value of the intensity of the pressure, and q_0 for that value when it (q) is constant. The small force acting outward on the differential of the arc $r_2 \Theta'$, or FAB , Fig. 18, is— $q r_2 d\Theta'$.

The distance from the center of the circle of contact to the center of the section, when R is the distance from the center to the exterior point, is $\frac{R + r_2}{2}$. Hence the lever arm of the small force given above is:—

$$\frac{R + r_2}{2} \sin. (\Theta - \Theta') \dots\dots\dots (44)$$

Putting $d(M)$ for its moment, there results:—

$$d(M) = r_2 \frac{R + r_2}{2} \gamma_2 q \sin.(\Theta - \Theta') d\Theta' \dots \dots (45)$$

For *ND*, Fig. 18, $q = q_0 = \text{constant}$ in Eq. 45, and for the arc *FN*, $q = \frac{\Theta'}{\beta} q_0$. Substituting these two values separately in Eq. 45, and putting the two parts together:—

$$M' = \frac{r_2 q_0}{\beta} \frac{R + r_2}{2} \int_0^\beta \sin.(\Theta - \Theta') \Theta' d\Theta' + q_0 r_2 \frac{R + r_2}{2} \int_\beta^\Theta \sin.(\Theta - \Theta') d\Theta' \dots \dots (46)$$

$$\therefore M' = q_0 r_2 \frac{R + r_2}{2} \left\{ 1 + \frac{\sin.(\Theta - \beta) - \sin.\Theta}{\beta} \right\} \dots (47)$$

Eq. 47 is applicable to any section for which Θ (locating the position of the section from *AF*, Fig. 18), is between α_2 and β . For the section *HK*, Fig. 19, α_2 is to be written for Θ , then remembering that $\sin.(\alpha_2 - \beta) = \sin.\delta$:—

$$M' = q_0 r_2 \frac{R + r_2}{2} \left\{ 1 - \frac{\sin.\alpha_2 - \sin.\delta}{\beta} \right\} \dots \dots (48)$$

When any section cuts *FN*, Fig. 18, the second part of the integral in Eq. 46, is to be omitted, and the first to be taken between the limits Θ and 0. These steps give:—

$$M' = r_2 \frac{R + r_2}{2} \frac{q_0}{\beta} \left\{ \Theta - \sin.\Theta \right\} \dots \dots (49)$$

Eq. 49 applies when Θ has any value between β and 0. If Θ is made equal to β in Eq. 49, and $\delta = 0$, (consequently $\beta = \alpha_2$) in Eq. 48, identical results will be obtained.

When q is not constant at any part of the surface of contact, but varies from the point where contact begins uniformly to the point of greatest compression, Eq. 49 is made applicable by simply replacing β by α_2 , so that:—

$$M' = q_0 r_2 \frac{R + r_2}{2} \left(\frac{\Theta - \sin.\Theta}{\alpha_2} \right) \dots \dots (50)$$

By inspecting Eqs. 47 to 50, it appears that M may be put under the general form

$$M' = q_0 r_2 \frac{R + r_2}{2} B \dots \dots (51)$$

The quantity B will, of course, be determined according to the location of the section under discussion by one of the Eqs. 47–50.

The bar tension acting in the section *DE*, Fig. 19, through L as a center, in the direction of the arrow tends to turn the part *DHKE* in a

direction opposite to that of the normal pressure, but in excess of it, as explained before. The lever arm of this tension about the center B , Fig. 19, of any section situated at the angular distance Θ from AC is:—

$$(r_1 + e) - \frac{r_2 + R}{2} \sin. (\alpha_2 - \Theta)$$

therefore the moment is:—

$$M'' = \frac{P}{2} (r_1 + e) - \frac{P}{2} \frac{r_2 + R}{2} \sin. (\alpha_2 - \Theta) \dots (52)$$

Consequently the resultant moment is:—

$$M = M'' - M' \dots (53)$$

The moment of resistance of the stress in the section is:—

$$M = \frac{(R - r_2)^2}{6} at \dots (54)$$

In Eq. 54, t is the actual intensity of the stress exerted on the fibres at the exterior of the section; $i. e.$, at the point furthest from the neutral axis, and at is the apparent maximum intensity of stress in the section. The constant a , involves the resistance which is offered to the fibres passing over each other, or longitudinal sheering. The experiments of Kirkaldy and others shows that the average value of a is about 1.6 for a square section, and would probably be a little larger for the deep rectangular section in front of the pin.

Substituting the values of M , M' and M'' in Eq. 53 (the value for M' is to be taken from Eq. 51), and solving for t , the following value is obtained:—

$$t = \frac{3 P (r_1 + e) - (3 R + r_2) \left\{ \frac{P}{2} \sin. (\alpha_2 - \Theta) + q_0 r_2 B \right\}}{a (R - r_2)^2} \dots (55)$$

There is but one section to which the bending force acts parallel and that one is HK , Fig. 19. The force which acts on any other section has one component, normal to it, acting as a direct tensile stress uniformly distributed, or in other words the intensity of this stress in the section is constant. The total force acting on the section AB , in the direction of the axis of the bar is the difference between the tension in the section DE , Fig. 19, and the axial component of the normal pressure on the arc GF . This force has one component normal to the section considered and one parallel to it. The former acts as a direct pull, and the latter, of course, is a direct sheering force.

Let q be the general expression for the intensity of the common pressure at any point, and the other data as before used and explained at Fig. 18. Suppose AH , to correspond to B , Fig. 19. The value of the pres-

sure exerted on an indefinitely small portion of any arc, as BF , Fig. 18, at B , is $r_2 q d\Theta'$. Each of these small normal forces is to be resolved in the manner shown at B into components normal to AH and parallel to the axis of the bar, or AD , Fig. 18. In that figure BK is perpendicular to AH , and KC parallel to AD ; also BC stands for $r_2 q d\Theta'$ and KC for its axial component. Putting F for the sum of these components, there results:—

$$d(F) = r_2 q \sin. (\gamma + \Theta') d\Theta' + r_2 q \cot. (\gamma + \Theta) \cos. (\gamma + \Theta') d\Theta' \dots (56)$$

For the first value of Θ' , take the section so that Θ shall lie between β and α_2 , then remembering that between Θ and β , $q = q_0 = \text{constant}$, and for the values Θ to β , $q = q_0 \frac{\Theta'}{\beta}$:—

$$F = r_2 q_0 \int_{\beta}^{\Theta} \left\{ \sin. (\gamma + \Theta) d\Theta' + \cot. (\gamma + \Theta) \cos. (\gamma + \Theta') d\Theta' \right\} + \frac{r_2 q_0}{\beta} \int_0^{\beta} \left\{ \sin. (\gamma + \Theta) \Theta' d\Theta' + \cot. (\gamma + \Theta) \cos. (\gamma + \Theta') \Theta' d\Theta' \right\} \dots (57)$$

$$\therefore F = \frac{r_2 q_0}{\beta} \left\{ \sin. (\gamma + \beta) - \sin. \gamma + \cot. (\gamma + \Theta) (\cos. (\gamma + \beta) - \cos. \gamma) \right\} \dots \dots \dots (58)$$

When $\Theta = \alpha_2$, $\cot. (\gamma + \alpha_2) = \cot. 90^\circ = 0$:—

$$F = \frac{r_2 q_0}{\beta} \left\{ \sin. (\gamma + \beta) - \sin. \gamma \right\} \dots \dots \dots (59)$$

If Θ is less than β , the first part of the integral expression for F in Eq. 57 is to be omitted, then :—

$$F = \frac{r_2 q_0}{\beta} \left\{ \sin. (\gamma + \Theta) - \sin. \gamma + \cot. (\gamma + \Theta) [\cos. (\gamma + \Theta) - \cos. \gamma] \right\} \dots \dots \dots (60)$$

Should q not be constant at any part of the surface of contact, α_2 is to be written for β in Eq. 60 :—

$$F = \frac{r_2 q_0}{\alpha_2} \left\{ \sin. (\gamma + \Theta) - \sin. \gamma + \cot. (\gamma + \Theta) [\cos. (\gamma + \Theta) - \cos. \gamma] \right\} \dots \dots \dots (61)$$

In Eq. 61, for the section HK , Fig. 19, $\Theta = \alpha_2$ and

$\cos. (\gamma + \alpha_2) = 0$; also $\sin. (\gamma + \alpha_2) = 1$, therefore :—

$$F = \frac{r_2 q_0}{\alpha_2} (1 - \cos. \alpha_2) \dots \dots \dots (62)$$

Putting P' for the total force exerted on the section parallel to the axis of the bar, there results :—

$$P' = \frac{1}{2} P - F \dots \dots \dots (63)$$

F will of course be determined by one of the previous equations.

Now P' has a component normal to the section situated at the angular distance ϕ from AC , Fig. 19, whose value is

$$P' \sin. (\alpha_2 - \phi) \dots \dots \dots (64)$$

The area of the section, since the thickness of the head is supposed to be unity, is $(R - r_2)$. If t'' stands for the intensity of the stress, then as it is constant :—

$$t'' = \frac{P' \sin. (\alpha_2 - \phi)}{R - r_2} \dots \dots \dots (65)$$

In order to complete the number of tensile stresses acting in any section there remains but to find that one of the principal internal stresses whose direction is around the pin-hole, the total amount of which is equal to the normal component of the pressure. The law, as previously shown, which governs this internal stress is the same as that for a closed cylinder whose interior radius is $r_2 A$. The quantity A is given by some one of Eqs. 24, 25 and 26.

The intensity of the tensile stress at the exterior point of any section in front of the pin is, from Eq. 6 :—

$$t = \frac{2 q_0 r_0^2}{R_0^2 - r_0^2} \dots \dots \dots (66)$$

R_0 and r_0 being the exterior distance and interior radius for the imaginary link head. In Eq. 66, r_0 must be replaced by $r_2 A$, and R_0 by $R - (1 - A) r_0$ in order that the actual quantities sought may be involved. Making the substitutions in Eq. 66 :—

$$t' = \frac{2 q_0 r_2^2 A^2}{R^2 - R(1 - A) 2 r_2 + r_2^2 (1 - 2 A)} \dots \dots \dots (67)$$

In Eq. 67, q_0 is of course the intensity of the normal pressure at the point where the section cuts the surface of contact. For intermediate points, Eqs. 2 and 3 must be used.

In order to get the tensile stress at any point in the head, it is only necessary to add the three intensities t , t' and t'' . The point at which these three values are to be taken is where the section under discussion cuts the exterior surface of the link head. At the surface of contact, rupture will take place by crushing, or combined crushing and shearing, so that to make a link head of uniform strength, it is necessary to so

proportion it, as to reach the ultimate compressive resistance of the material at the interior surface and at the same instant that the metal on the outside of the head is brought to its ultimate tensile resistance. This statement holds, of course, only when compression and tensile ruptures, in front of the pin, are to be provided for, and it will afterwards be seen that if these are guarded against, other contingencies will hardly arise. Let T be put for the intensity of the ultimate tensile resistance of the material on the exterior surface of the link head, and q_0 for that of the ultimate compressive resistance at the surface of contact. The preceding condition of uniformity of strength requires that the bending stress be taken as tensile, and that T shall equal the sum of the intensities given by Eqs. 55, 65 and 67, then

$$T = t + t' + t'' \dots \dots \dots (68)$$

Substituting for the values of t , t' and t'' there results:—

$$T = \frac{3P(r_1 + e) - 3(R + r_2) \left\{ \frac{P}{2} \sin. (\alpha_2 - \theta) + q_0 r_2 B \right\}}{\alpha (R - r_2)^2} + \frac{2q r_2^2 A^2}{R^2 - R(1 - A)2r_2 + r_2^2(1 - 2A)} + \frac{P' \sin. (\alpha_2 - \theta)}{R - r_2} \dots (69)$$

If it be required to give R such a value, that the tensile and compressive resistances of the metal on the surface of contact shall reach their ultimate values at the same time, the bending stress must be considered compression, having a sign different from that of t' and t'' ; and t' will be taken from Eq. 4.

If the value of R be thus calculated it will be found exceedingly small, as might easily be anticipated, so Eq. 69 is the only one that will be employed in determining R . The reason for this small value of R , is found in the existence of the bending compression at the surface of contact.

In order to simplify the reduction of Eq. 69 put—

$$\frac{P}{2} \sin. (\alpha_2 - \theta) + q_0 r_2 B = K;$$

$$3P(r_1 + e) = M,$$

$$\text{and } P' \sin. (\alpha_2 - \theta) = N;$$

then partially clearing fractions:—

$$R^2 \alpha T - 2 \alpha R r_2 T + \alpha r_2^2 T - M + 3RK + 3r_2 K - \alpha RN + \alpha r_2 N - \frac{2R^2 \alpha r_2^2 A^2 - 4R \alpha r_2^3 A^2 + 2 \alpha r_2^4 A^2}{R^2 - 2Rr_2(1 - A) + r_2^2(1 - 2A)} = 0 \dots (70)$$

$$\text{Put } (2 \alpha r_2 T - 3K + \alpha N) = C,$$

$$\text{and } (\alpha r_2^2 T - M + 3r_2 K + \alpha r_2 N) = D$$

in Eq. 70, then clearing away the fraction and arranging the terms according to the ascending powers of R : —

$$\begin{aligned} R^4 \alpha T - R^3 \left\{ C + \alpha T (1 - A) 2r_2 \right\} + R^2 \left\{ D + 2C(1 - A) r_2 + \right. \\ \left. \alpha T r_2^2 (1 - 2A) - 2 \alpha q r_2^2 A^2 \right\} - R \left\{ 2D(1 - A) r_2 + C r_2^2 (1 - 2A) - \right. \\ \left. 4 \alpha q r_2^3 A^2 \right\} - r_2^2 \left\{ 2 \alpha q r_2^2 A^2 - D(1 - 2A) \right\} = 0 \dots (70) \end{aligned}$$

On account of the great length of this equation, it will be best to use Eq. 70 for the purpose of substituting numerical values ; then, after that is done, the reduction may be much more simply concluded by clearing away the fractions and arranging in reference to R . An equation of the fourth degree will be obtained, which may be quite easily solved by trial without resorting to the methods of higher algebra.

One element of uncertainty exists in Eq. 70 as well as in all others. The quantity q_0 for metal situated like that in front of the pin, is unknown. It is altogether probable that this intensity is a high one, as the material is well supported, but it yet remains to be determined by experiment, and Eq. 70 can not be accurately applied until this quantity is known.

The dimensions of a link head will be determined by the preceding formulæ, in order to show their mode of application and to test their correctness. In the first place, the pressure of contact (normal to the surface) will be assumed to vary uniformly from the axis of the bar around to the point where contact begins, or what is the same thing, the material of the head is considered to be perfectly elastic under all intensities of stress.

This has an important bearing on the actual case which will be seen hereafter.

Example I. The method of approximation to be pursued, is that pointed out between Eqs. 39 and 40. After fixing the data, trial values of α_2 are placed in Eqs. 38 and 39, and values of q_0 are deduced therefrom. When these last agree, the proper value of α_2 has been taken. Before this operation can be performed, a value of R must be inserted in Eq. 38 ; fortunately this quantity so enters that equation that a considerable error in its value will not appreciably affect the result. It will be sufficient to put R equal to $r + 1\frac{1}{8}$, the width of the bar.

Suppose that the pin is 0.02 inches smaller than the pin-holes, the width of bar, $W = 4$ inches ; the thickness = 1 inch ; $r = 1.75$ $r_1 = 1.76$ and $R = 4.5$ inches ; $E = 14\,000$ tons ; $\alpha_2 = 50^\circ$.

$\alpha_2 = 0.87\ 266$; $\sin. \alpha_2 = 0.76\ 604$; $\cos. \alpha_2 = 0.64\ 279$, and $P = Wt = 4 \times 15 = 60$ tons.

Substituting in Eq. 38 and reducing :—

$q_0 (0.283\ 724 + 0.0\ 301) = 12.5 = 0.313\ 824\ C_0$; $\therefore q_0 = 40$ tons.
From Eq. 39, $q_0 = 42$ tons; hence, α_2 was taken too small. Making α_2 equal to 51° ;—from Eq. 38, $q_0 = 42.4$ tons, and from Eq. 39, $q_0 = 41.1$ tons.

Therefore α_2 was taken a very little too large. Comparing the results, it is seen that the following is about the correct value of α_2 :

$$\alpha_2 = 50^\circ\ 45' \dots\dots\dots (71)$$

$$\therefore q_0 = 41.6 \dots\dots\dots (72)$$

The quantities now known substituted in Eq. 70 will give the distance KH , Fig. 6.

$$\text{Comparing Eqs. 26 and 50: } A = B = \frac{\alpha_2 - \sin. \alpha_2}{\alpha_2} = 0.1257.$$

Also, $e = 1.2$ inches, $T = 15$ tons, $\Theta = \alpha_2 = 50^\circ\ 45'$. q_0 is taken from Eq. 72. Substituting in Eq. 70 :

$$24R^2 - 56.5R - 411.0 = \frac{6.44R^2 - 22.5R + 19.7}{R^2 - 3.06R + 2.29}$$

$$\text{Solving by trial, } R = 5.5 \dots\dots\dots (73)$$

$$\therefore R - r = 5.5 - 1.75 = 3.75 = KH, \text{ (Fig 22).}$$

Example II. $r = 1.5$, $r_1 = 1.51$ and $e = 1.2$ inches; $E = 14\ 000$, $P = 60$ and $T = 15$ tons and $a = 1.6$.

Substituting in Eqs. 38 and 39, and solving by trial, these values are found : $q_0 = 48.2$ tons, $\alpha_2 = 50^\circ\ 50' \dots\dots\dots (74)$

For KH , Fig. 22, again take $\Theta = \alpha_2$, in Eq. 70, Eqs. 26 and 50 give $A = B = 0.126$.

$$\therefore 24R^2 - 44.66R - 392.0 = \frac{5.51R^2 - 16.53R + 12.4}{R^2 - 2.62R + 1.683}$$

$$\text{This equation is satisfied by } R = 5.1 \dots\dots\dots (75)$$

Example III. $r = r_2 = 2$ inches; $r_1 = 2.01$ and $e = 1.2$ inches; $E = 14\ 000$; $P = 60$ and $T = 15$ tons and $a = 1.6$.

Substitutions in Eqs. 38 and 39 give :—

$$q_0 = 36.15 \text{ tons, } \alpha_2 = 50^\circ\ 40' \dots\dots\dots (76)$$

As before $A = B = 0.1253$ and $N = 0$; make $\Theta = \alpha_2$ in Eq. 70, then substituting :—

$$24R^2 - 68.8R - 427.4 = \frac{7.265R^2 - 29.06R + 29.06}{R^2 - 3.5R + 3}$$

$$\therefore R = 5.92$$

$$KH, \text{ (Fig. 22)} = R - r_2 = 5.92 - 2 = 3.92 \dots\dots\dots (77)$$

If, in consequence of the nature of the material, the value of q_0 is limited so that it cannot exceed a certain amount, the preceding results will be somewhat modified.

In the actual case, after the intensity of the common, normal pressure has reached the limit of compressive resistance of material situated like that in front of the pin, any increase of tension in the bar will only serve to force the pin into the head without increasing the maximum intensity of the pressure between the surfaces of contact. For some distance on each side of the axis of the link, this intensity will remain constant at a value depending on the nature of the material; beyond, of course, it will vary down to zero, where contact ceases. The distance over which the intensity does not vary is not easy to determine exactly, but a close approximation may be obtained. Let the angular amount of contact be determined for a link head of perfectly elastic material, as in the preceding examples. The point at which exists an intensity equal to that of the ultimate compressive resistance of the material may then be easily determined. Suppose δ' to be this angular distance, measured from the longitudinal axis of the bar, HA , Fig. 19; also q_0 the limit of the resistance, and q_0' the maximum intensity of the pressure in front of the pin for a perfectly elastic head. Finally, put δ for the angular distance over which q_0 is constant in the actual case. Now if the *excess of pressure* which exists over the arc $r\delta'$, with an intensity varying from zero to $q_0' - q_0$, be distributed with the constant intensity q_0 over an arc x , with the radius r , the following equation will hold true:—

$$\frac{q_0' - q_0}{2} r\delta' = q_0 r x \therefore x = \frac{q_0' - q_0}{2 q_0} \delta' \dots \dots \dots (78)$$

From Eq. 78, the following approximate value of δ at once results:—

$$\delta' + x = \delta \dots \dots \dots (79)$$

This value will be a little smaller than the true one, and the error will be greater as δ' increases. In ordinary cases, however, the error is very small.

Before α_2 can be determined it is necessary to find P as a function of δ . All data will remain the same as before, β being equal to $\alpha_2 - \delta$, and measured from $A F$, Figs. 18, 19.

Put BC , Fig. 21, for $r_2 q d\phi$ and resolve it as shown by BMC . There results:—

$$MC = d \left(\frac{1}{2} P \right) = r_2 q d\phi \sin. (\theta + \gamma).$$

Also as before, $q = \frac{\Theta}{\beta} q_0$. First, let Θ , locating the section under consideration, be greater than β ; then remembering that $d\left(\frac{1}{2}P\right)$ consists of two parts, i.e., for q_0 and q :-

$$\frac{1}{2} P = r_2 q_0 \int_{\beta}^{\Theta} \sin. (\Theta + \gamma) d\Theta + \frac{r_2 q_0}{\beta} \int_0^{\beta} \sin. (\Theta + \gamma) \Theta d\Theta$$

$$\therefore \frac{1}{2} P = r_2 q_0 \left\{ \frac{\sin. (\beta + \gamma) - \cos. \alpha_2}{\beta} - \cos. (\Theta + \gamma) \right\} \dots (80)$$

If $\Theta = \alpha_2$, $\sin. (\beta + \gamma) = \cos. \delta$, and $\cos. (\alpha + \gamma) = 0$.

$$\frac{1}{2} P = \frac{r_2 q_0}{\beta} (\cos. \delta - \cos. \alpha_2) \dots \dots \dots (81)$$

If $\beta = \alpha_2$, Eq. 80 gives for $\Theta = \alpha_2$ -

$$\frac{1}{2} P = r_2 q_0 \frac{1 - \cos. \alpha_2}{\alpha_2} \dots \dots \dots (82)$$

a result identical with Eq. 39. If Θ is less than β , then:-

$$\frac{1}{2} P = \frac{r_2 q_0}{\beta} \int_{\beta}^{\Theta} \sin. (\Theta + \gamma) \Theta d\Theta$$

$$\therefore \frac{1}{2} P = \frac{r_2 q_0}{\beta} \left\{ \sin. (\Theta + \gamma) - \Theta \cos. (\Theta + \gamma) - \cos. \alpha_2 \right\} \dots (83)$$

$$\text{If } \Theta = \alpha_2 = \beta; \quad \frac{1}{2} P = r_2 q_0 \frac{1 - \cos. \alpha_2}{\alpha_2}$$

identical with Eqs. 39 and 82. After δ has been determined, Eqs. 80, &c., involve only the unknown term α_2 , but as the second members of those equations are transcendental functions, α_2 will be found by trial. The term β is to be replaced by $(\alpha_2 - \delta)$. When a trial value of α_2 has been so chosen that the second members of the equations above are equal to the first ($\frac{1}{2}P$ a known quantity) then that value is the correct one, and Eq. 70 may at once be employed.

The intensities of the radial pressure and stress around the pin at any point in the head are found at once from Eqs. 2 and 3; remembering to first put $R - (1 - A)r$ for R , and Ar for r_0 , also $r^1 = (1 - A)r$ for r^1 in those formulæ.

The intensity of the direct stress on any section is found by Eq. 65, and that of the maximum bending by Eq. 55. Eq. 67 gives the intensity of the stress around the pin at a point farthest from the centre, in any section. In the following calculations for the dimensions of two actual eye-bar heads, q_0 will be taken at 25 tons, and T at 15 tons as before. The value of q_0 is not known from any experiments, and it is taken rather high in consequence of the support which the metal in front of the pin receives.

Example IV.—All data are the same as in *Example I*. From the equation, $q = \frac{\Theta}{\beta} q_0$, the value of δ' for Eq. 78 is found to be 20.3° . The data gives $q'_0 = 41.6$, and $q_0 = 25$. $\therefore x = 6.74^\circ$. Since this is a little too small, take $x = 7^\circ$. then

$$\delta = x + \delta' = 26^\circ 18'.$$

Substituting this in Eq. 80, and solving by trial :

$$\alpha_2 = 60^\circ. \dots\dots\dots (84)$$

$$\text{Hence } \beta = 60^\circ. - 26^\circ 18' = 33^\circ 42'$$

Let the first section, for discussion, be taken immediately in front of the centre of the pin, for which $\Theta = \alpha^2$; making $\Theta = \alpha_2$ in Eq. 24, $A = 0.3$.

Eq. 48 determines B , and N in Eq. 70 is equal to zero. Making substitutions in Eq. 70 :—

$$24 R^2 - 44.66 R - 390.4 = \frac{19.3 R^2 - 67.5 R + 59.1}{R^2 - 2.45 R + 1.23}$$

Solving by trial :—

$$R = 5.11 \therefore R - r_2 = 3.36 \dots\dots\dots (85)$$

For three different stresses :— $t = 14.64$, $t' = 0.43$ tons, and $t'' = 0$.

$\therefore T = 15.07$ tons.

If $\Theta = 34^\circ$, Eq. 24 gives $A = 0.1964$; substituting in Eq. 70 :—

$$24 R^2 - 49.05 R - 359.2 = \frac{8.28 R^2 - 29 R + 25.35}{R^2 - 2.8 R + 1.86}$$

$$\therefore R = 5.06 \text{ and } R - r_2 = 3.31 \dots\dots\dots (86)$$

Also, $t = 12.1$, $t' = 0.48$, and $t'' = 2.1$ tons; $\therefore T = 14.7$ tons.

Next let $\Theta = 18^\circ$. Eq. 25 gives $A = 0.083$, substituting in Eq. 70 :—

$$24 R^2 - 51.64 R - 301.1 = \frac{0.9142 R^2 - 3.2 R + 2.8}{R^2 - 3.21 R + 2.55}$$

$$\therefore R = 4.8 \text{ and } R - r_2 = 3.05 \dots\dots\dots (87)$$

Then, $t = 8.8$, $t' = 0.13$, and $t'' = 6$ tons. $\therefore T = 14.93$ tons.

Finally, let $\Theta = 0$, $A = 0$, $B = 0$ and $P = \frac{1}{2} P$. \therefore Eq. 70 gives,

$$R^2 - 2 R = 10.42.$$

$$\therefore R = 4.38 \text{ and } R - r_2 = 2.63 \dots\dots\dots (88)$$

Whence, $t = 5$, $t' = 0$, and $t'' = 9.9$ tons $\therefore T = 14.9$ tons.

For any section, NO , Fig. 19, back of the centre of the pin, it is to be considered that the only stresses existing are the bending stress, caused by the tension distributed over the section DE , and acting through the centre L , and t'' resulting from the direct pull on the section in question.

Therefore, Eq. 88 gives the value of R for a section whose angular distance from AS , Fig. 22, is 60° . Eq. 70 will apply to any other section back of the centre of the eye by putting α_2 for its angular distance from the axis of the bar, and taking $\Theta = 0$.

Suppose $r_2 = 1.75$ and $r_1 = 1.76$ inches; $\alpha_2 = 40^\circ$ and $\Theta = 0$, therefore, $A = 0 = B$, and $P' = \frac{1}{2} P = 30$ tons.

Applying Eq. 70:—

$$R^2 - 2.375 R = 12.666 \therefore R = 4.95 \dots \dots \dots (89)$$

And $R - r_2 = 3.2$; also, $t = 8.85$, $\ell = 0$ and $\ell' = 6.03$ tons;
 $\therefore T = 14.9$ tons.

Example V. $r_2 = 1.5$, $r_1 = 1.51$, $W = 4$ and $e = 1.2$ inches;
 $a = 1.6$, $T = 15$, $P = 60$, $q_0 = 25$ and $q'_0 = 48.2$ tons (*Example II*);
 also $\delta' = 24.5^\circ$.

Eq. 78 gives $x = 11.35^\circ$, therefore, $\alpha_2 = 75^\circ 45'$; $\gamma = 14^\circ 15'$ and
 and $\beta = 41^\circ 15'$

First take the section immediately in front of the pin, for which
 $\Theta = \alpha_2$;—

$$24 R^2 - 22.5 R - 360 = \frac{34.9 R^2 - 104.5 R + 78.41}{R^2 - 1.68 R + 0.27}$$

$$\therefore R = 4.5; R - r_2 = 3 \dots \dots \dots (90)$$

Therefore, $t = 13.2$, $\ell = 1.7$ and $\ell' = 0$ tons; $\therefore T = 14.9$ tons.

Next take $\Theta = \beta = 41^\circ 15'$.

$$24 R^2 - 24.52 R - 323.6 = \frac{18.6 R^2 - 55.64 R + 41.73}{R^2 - 2.037 R + 0.806};$$

$$\therefore R = 4.3, \text{ and } R - r_2 = 2.8 \text{ inches } \dots \dots \dots (91)$$

Then, $t = 10.9$, $\ell = 1.1$, $\ell' = 3$ tons; $\therefore T = 15$ tons.

Suppose $\Theta = 20^\circ$.

$$24 R^2 - 28.8 R - 272 = \frac{1.55 R^2 - 4.66 R + 3.5}{2 - 2.6 R + 1.65}$$

$$\therefore R = 4.02 \text{ and } R - r_2 = 2.52 \text{ inches } \dots \dots \dots (92)$$

Consequently $t = 6.93$, $\ell = 0.2$ and $\ell' = 8$ tons.

$$\therefore T = 15.13 \text{ tons.}$$

For $\Theta = 0$, $24 R^2 - 31.26 R - 233.1 = 0$.

$$R = 3.83 \text{ and } R - r_2 = 2.33 \text{ inches } \dots \dots \dots (93)$$

$\therefore t = 2.55$, $\ell = 0$ and $\ell' = 12.5$ tons; $\therefore T = 15.05$ tons.

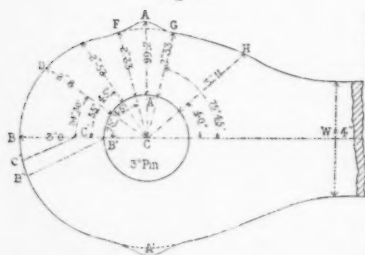
In order to get R for sections back of the center of the pin, the method to be observed is that given in *Example IV*.

Take $\alpha_2 = 40^\circ$ and $\Theta = 0$; $\therefore 24 R^2 - 45 R - 300.7 = 0$.

$\therefore R = 4.61$ and $R - r_2 = 3.11$ inches..... (94)

Then, $l = 8.7$, $l' = 0$ and $l'' = 6.2$ tons. $\therefore T = 14.9$ tons.

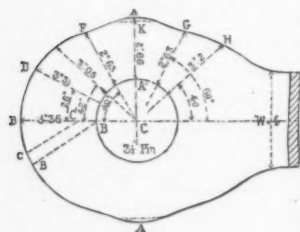
Fig. 20.



The data of *Example V*, are used in the construction of Fig. 20, and those of *Example IV*, in Fig. 21. The positions of the dimensions on the figures render a detailed explanation of the construction unnecessary.

The swellings at the points AA' , although presenting a singular appearance are easily explained. The existence of a point of contra-flexure in the vicinity of F , has been previously indicated and its position is to be found according to the conditions of Eq. 43. But in this case, Fig. 23, $r_1 + e = 2.7$ inches, is larger than $R = 2.66$ inches at an angular distance of $14^\circ 15'$ from AA . This simply shows, that in the search by approxi-

Fig. 21.



mation for the value of $R - r_2$ at F , the number taken for R should have been a very little larger than 3.83.

This fact is confirmed by the value of T , immediately following Eq. 93. So that after the small errors of approximation are eliminated, the only cause for the swelling at A , is the bending moment in AA'' , on account of the con-

dition of imperfect flexibility in that section.

The slight increase just pointed out as belonging to 2.33 inches, would, of course, make the depression at F , less than that shown in Fig. 20. The corresponding number in the approximation for Fig. 21 was better chosen, and the diagram is nearly correct.

It will be remembered that AA'' , Figs. 20 and 21, is equal to 0.666 W , and that the intensity of stress at A'' is the same as that in the body of the bar i. e., 15 tons; and experiment has demonstrated the safety of that proportion of W .

The capacity of the metal to stretch, affects in a very marked manner the quantity of it in the section AA' , as will be shown by the results of

the following investigation. It will be supposed that the pressure is applied with constant intensity in front of the pin, but varying to zero on each side of this surface. Suppose in Fig. 22 that EFG is half the interior surface of the pin hole, and that half which lies in front of the pin; also that BH lies in the axis of the bar. Let contact



between the pin and head begin at C , then put $FHC = \alpha_2$, $FHD = \delta$, $DHC = \beta$, and $CHE = \gamma$.

These letters carry the same signification as when used before, that is, $q = q_0$ is constant over FD , and q is variable over DC . Let EA stand for $\frac{1}{2} AA''$ in Figs. 20 and 21, and take ABK as a circle, supposing all the metal of the head concentrated in it. Put $AH = R$ and $HE = r$. A little consideration will show that the case here presented is that of an arched rib, with ends fixed and parallel.

The condition of fixedness is obtained by a moment at A , tending to bend in the same direction as the moment of the forces applied between F and C . The value of this moment is to be sought, for when it is once obtained, the centres of stress for the sections AA'' , Figs. 20 and 21, can easily be determined, and therefore, the widths of the sections themselves. Let x and y be the abscissa and ordinate of vertical deflection of any point in the semicircle AB , considered positive to the left and upward, the origin being at A , Fig. 22. If M stands for the moment of the forces applied to any beam about any section, then the formula, $M = EI \frac{d^2 y}{dx^2}$ holds good, whether that beam be curved or straight, inclined or horizontal. Suppose γ' to stand for any angle less than γ measured from AH , and Θ for any angle measured from HC to HF , Fig. 22, also— M , the negative moment at A . $\frac{1}{2} P$ pulls directly back at

A . Then for any point between A and C' :—

$$\frac{1}{2} PR (1 - \cos. \gamma') - M' = EI \frac{d^2 y}{dx^2}$$

$$\text{But } x = R (1 - \cos. \gamma') \therefore dx^2 = R^2 \sin.^2 \gamma' d\gamma'^2 \therefore$$

$$\frac{1}{2} P R (1 - \cos. \gamma') - M' = EI \frac{d^2 y}{R^2 \sin.^2 \gamma' d\gamma'^2} \dots \dots \dots (95)$$

Integrating :—

$$\frac{PR^2}{4} (1 - \cos. \gamma')^2 M' R (1 - \cos. \gamma') = \frac{dy}{R \sin. \gamma' d\gamma'} EI \dots (96)$$

In Eq. 96, the constant of integration is equal to 0. If $y' = y$,

$$EI \frac{dy}{R \sin. \delta' dy'} = EI \tan. y'.$$

$$\frac{P R^2 (1 - \cos. y')^2}{4} - M' R (1 - \cos. y) = EI \tan. y' \dots \dots (97)$$

Since the ends are fixed in position at *A* and *K*, Fig. 22, the value of $\frac{dy}{R \sin. y' dy'}$ for $y' = 0$ will be zero. For the arc *DC*, Fig. 22,

$$X = R \left\{ 1 - \cos. (y + \epsilon) \right\} \therefore dx^2 = R^2 \sin.^2 (y + \epsilon) d\epsilon^2.$$

Since q is not constant for *DC* the moment of the applied pressure around any point in it will be given by Eq. 49, and will be negative. In this instance ϵ will only vary between 0 and β . Taking moments about any point of *DC*:-

$$\frac{P}{2} R \left\{ 1 - \cos. (y + \epsilon) \right\} - M' - q_0 r \frac{R}{\beta} - (\epsilon - \sin. \epsilon) = EI \frac{d^2 y}{R^2 \sin.^2 (y + \epsilon) d\epsilon^2} \dots \dots (98)$$

Integrating:-

$$EI \frac{dy}{R \sin. (y + \epsilon) d\epsilon} = \frac{P}{2} R^2 \left\{ -\cos. (y + \epsilon) - \frac{1}{2} \sin.^2 (y + \epsilon) \right\} + M' R \cos. (y + \epsilon) - q_0 r \frac{R^2}{2} \left\{ -\epsilon \cos. (y + \epsilon) + \sin. (y + \epsilon) - \frac{1}{2} \sin. y \sin.^2 \epsilon - \cos. y \left(\frac{\epsilon}{2} - \frac{\sin. 2\epsilon}{4} \right) \right\} + C \dots \dots (99)$$

Take Eq. 99 between the limits β and y , putting

$$\frac{dy}{R \sin. (y + \beta) d\beta} = \tan. \beta, \text{ and } \frac{dy}{R \sin. 2y dy} = \tan y,$$

then there results:-

$$EI \tan. \beta - EI \tan. y = \frac{P R^2}{2} \left\{ \cos. 2y - \sin. \delta - \frac{\cos.^2 \delta}{2} + \frac{\sin.^2 2y}{2} \right\} + M' R \left\{ \sin. \delta - \cos. 2y \right\} - q_0 r \frac{R^2}{\beta} A \dots (100)$$

In which equation:-

$$A = y \cos. 2y - \beta \sin. \delta - \sin. 2y \left(1 + \frac{\sin. \alpha_2}{4} \right) + \cos. y + \frac{\sin. y}{2} \left(\sin.^2 y - \sin.^2 \beta \right) - \sin. \alpha_2 \left(\frac{\beta - y}{2} - \frac{\sin. 2\beta}{4} \right). (101)$$

Over the part *F*, $Dq = q_0 = \text{constant}$, and the moment (negative) of the applied pressure about any point in the arc *FD* is given by Eq. 47.

Taking moments about any point of *FD*:-

$$EI \frac{d^2 y}{R^2 \sin.^2 (\gamma + \Theta) d \Theta^2} = \frac{PR}{2} \left\{ 1 - \cos. (\gamma + \Theta) \right\} - M' - q_0 r \frac{R}{\beta} \left\{ \beta + \sin. (\Theta - \beta) - \sin. \Theta \right\} \dots (102)$$

Integrating Eq. 102 between the limits α_2 and β , there results:—

$$EI \tan. \beta = \frac{P R^2}{2} \left\{ \frac{1}{2} - \cos. (\gamma + \beta) - \frac{1}{2} \sin.^2 (\gamma + \beta) + M' R \cos. (\gamma + \beta) - q_0 r \frac{R^2}{\beta} A' \dots \dots \dots (103) \right.$$

In which equation;—

$$A' = \left\{ \frac{\alpha_2 - \beta}{2} + \frac{\sin. 2 \beta - \sin. 2 \alpha_2}{4} \left\{ \cos. \gamma - \cos. (\beta - \gamma) \right\} + \left\{ \frac{1}{2} \sin. (\beta - \gamma) + \frac{1}{2} \sin. \gamma \right\} \left\{ \sin.^2 \alpha_2 - \sin.^2 \beta \right\} + \delta \sin. \beta \sin. \gamma - \beta \sin. \delta \dots \dots \dots (104) \right.$$

If γ exceeds β , the term $-\sin. (\beta - \gamma)$ in Eq. 104, must be replaced by $+\sin. (\gamma - \beta)$; but $\cos. (\beta - \gamma)$ will not be affected, since $\cos. (\beta - \gamma) = \cos. (\gamma - \beta)$.

Substituting from Eqs. 97 and 103 in Eq. 100:

$$M \left\{ \cos. \gamma - \cos. 2 \gamma - 1 \right\} = \frac{PR}{2} \left\{ \frac{1}{2} \cos. (\beta + \gamma) [\cos. (\beta + \gamma) - 2] - \frac{(1 - \cos. \gamma)^2}{2} - \cos. 2 \gamma + \sin. \delta + \frac{\cos.^2 \delta}{2} - \frac{\sin.^2 2 \gamma}{2} \right\} + q_0 r \frac{R}{\beta} \left\{ A - A' \right\}$$

$$\text{Or— } M'H = R \left\{ \frac{P}{2} A'' + \frac{q_0 r}{\beta} (A - A') \right\} = RA''' \dots (105)$$

The values of A'' , A''' and H , are evident from the equation preceding Eq. 105.

Let W' stand for the width of the the section AA'' , Figs. 7 and 8, then:—

$$M' = \frac{R A'''}{H} = \frac{W'^2 \alpha t}{6} \dots \dots \dots (106)$$

The same signification is given to α and t as was given when they were used before.

Putting $R = r + \frac{1}{2} W'$:—

$$t = \frac{6 \left(r + \frac{1}{2} W' \right) A'''}{W'^2 \alpha H} \dots \dots \dots (107)$$

This value of t , is the intensity of the tensile bending stress at A'' , Figs. 20 and 21, and the same is compression at A . The direct pull of

$\frac{1}{2} P$ will be distributed uniformly over the section W' with an intensity equal to $\frac{P}{2W'}$ and this is to be added to t , in order to be equal to T , the maximum intensity of the tensile stress in the section at A :

$$\therefore \frac{P}{2w'} + t - T = 0.$$

Substituting from Eq. 107, and reducing:—

$$W' = \frac{P\alpha H + 6A''}{4\alpha HT} - \sqrt{\left(\frac{P\alpha H + 6A''}{4\alpha HT}\right)^2 + \frac{6rA''}{\alpha HT}} \dots\dots (108)$$

The case in which q is not constant, but varies from C to F , Fig. 21, is treated by replacing β with a_2 in Eq. 99. Taking the resulting equation between the limits a_2 and γ :

$$\begin{aligned} EI \tan. \gamma &= \frac{PR^2}{2} \left\{ \frac{1}{2} - \cos. 2\gamma - \frac{1}{2} \sin.^2 2\gamma \right\} + MR \cos. 2\gamma - \\ q_0 r \frac{R^2}{\alpha_2} &\left\{ -1 + \frac{1}{2} \sin. \gamma \cos.^2 \gamma + \cos. \gamma \left(\frac{\alpha_2}{2} - \sin.^2 \frac{\alpha_2}{4} \right) - \right. \\ \gamma \cos. 2\gamma &+ \sin. 2\gamma - \frac{1}{2} \sin.^3 \gamma - \cos. \gamma \left(\frac{\gamma}{2} - \frac{\sin. 2\gamma}{4} \right) \left. \right\} \dots (109) \end{aligned}$$

Subtracting Eq. 97 from Eq. 109:—

$$MR (1 + \cos. 2\gamma - \cos. \gamma) = R^2 F \dots\dots\dots (110)$$

$$\begin{aligned} \text{Whence, } F &= \frac{P}{2} \left\{ \frac{(1 - \cos. \gamma)^2}{2} + \cos. 2\gamma + \frac{\sin.^2 2\gamma}{2} - \frac{1}{2} \right\} + \\ q_0 \frac{\gamma}{\alpha_2} &\left\{ \frac{\sin. \gamma \cos.^2 \gamma}{2} - 1 + \cos. \gamma \left(\frac{\alpha_2 - \gamma}{2} - \frac{\sin. 2\alpha_2}{4} - \frac{\sin. 2\gamma}{4} \right) - \right. \\ &\left. r \cos. 2\gamma + \sin. 2\gamma - \frac{1}{2} \sin.^3 \gamma \right\} \end{aligned}$$

$$\therefore M' = \frac{RF}{1 + \cos. 2\gamma - \cos. \gamma} = \frac{RF}{H} \dots\dots\dots (111)$$

$$\text{But— } M' = \frac{W'^2}{6} \cdot a t = \frac{\left(r + \frac{1}{2} w'\right) F}{H}$$

$$\therefore t = \frac{6 \left(r + \frac{1}{2} W'\right) F}{W'^2 H \alpha} \dots\dots\dots (112)$$

If T represents the maximum intensity of tensile stress which it is desirable to impress upon the metal:—

$$\frac{P}{2W'} + \frac{6rF + 3W'F}{W'^2 \alpha H} - T = 0$$

$$\therefore W' = \frac{\alpha PH + 6F}{4\alpha HT} + \sqrt{\left(\frac{\alpha PH + 6F}{4\alpha HT}\right)^2 + \frac{6rF}{\alpha HT}} \dots (113)$$

Applying Eqs. 106, 107 and 108, to the case of *Example IV*, there results:

$$W' = AA'', (\text{Figs. 22, 23}) = 4.835 \text{ inches...} \quad (114)$$

$$M' = 55.04 \text{ inch-tons...} \quad (115)$$

$$t = 8.81 \text{ tons...} \quad (116)$$

$$T = \frac{P}{2W'} + t = +15 \text{ tons or } -2.6 \text{ tons...} \quad (117)$$

The last result shows that on the outside of the head, at A , (Figs. 20, 24), compression exists with an intensity of 2.6 tons. The equation named above when applied to *Example V*, gives :

$$W' = 4.17 \text{ inches...} \quad (118)$$

$$M' = 36.2 \text{ inch-tons...} \quad (119)$$

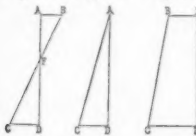
$$t = 7.8 \text{ tons...} \quad (120)$$

$$\therefore T = \frac{P}{2W'} + t = +15 \text{ or } -0.6 \text{ tons...} \quad (121)$$

These show that on the outside of the head, the intensity of the compression is 0.6 tons. The results reveal a great discrepancy, when compared with the figures which experiments show to be sufficient to give the section AA'' , Figs. 20 and 21, an ultimate strength equal to that at any other part of the head. In this respect, the results of the last analysis are exceedingly unsatisfactory, for it is evident that the formulæ call for an amount of metal at AA'' far in excess of that actually needed; yet they are very valuable as showing the true condition of affairs existing in the section AA'' . It is true that there are one or two sources of error in the investigation—such as considering the metal to be of uniform width in front of the pin and concentrated in a semicircle (also omitting the direct extension of the metal), but the resulting errors must be very small and cannot account for the discrepancy.

Whatever amount of material is necessary on a line through the centre of the pin and perpendicular to the axis of the bar, in order to give the proper degree of strength, it is apparent that it must be determined by experiments carefully conducted, and for this reason a somewhat arbitrary value of the quantity e , was taken in the examples that have been worked out.

Fig. 23. Fig. 24. Fig. 25. The value of e is easily obtained when M' is known, and consequently the intensities of the stresses at A and A'' , Figs. 19 and 20. The only three cases which can exist are represented by Figs. 23, 24, 25. Let CD , AB and all lines parallel to them be abscissæ, and $AD = W'$.



Therefore, C being the origin, the abscissæ represent intensities, and the ordinates, distances on W' . Fig. 23, is the case in which compression exists on the outside of the head; Fig. 24, that in which the stress at that point is zero, and Fig. 25, that which occurs when BA is the same kind as CD .

The distance of the centre of stress from D is to be found in each case; let it be denoted by e . First take Fig. 23. Put $DF = a$, and $FA = b$. Similar triangles give: $\frac{B}{W'} = \frac{-T'}{T - T'}$. $\therefore b = -W' \frac{T}{T - T'}$. It is to be noticed that, $AD = W'$, $CD = T$, and $AB = T'$. Then—

$$a = W' - B = W' \left(1 + \frac{T'}{T - T'} \right).$$

The difference of the areas of the triangle CDF and $ABF = \frac{P}{2}$, hence by static moments:—

$$\frac{1}{2} P e = \int_0^a \frac{T'}{a} (a - x) x dx + \int_0^b \frac{T'}{b} (a + x) x dx,$$

$$\therefore e = \frac{1}{P} \left[\frac{1}{3} a^2 T' + b T' \left(a + \frac{2}{3} b \right) \right]$$

As an example, let $P = 60$ tons, $W' = 4.84$ inches, $T = 15$ and $T' = -2.6$ tons.

Making the substitution; $a = 4.12$, $b = 0.71$, and $e = 1.27$.

In the case of Fig. 24, it is evident that the centre of stress is one-third the distance from D to A , therefore, $e = \frac{1}{3} W'$.

Fig. 25 must be treated similarly to Fig. 26. Taking static moments about D , there results:—

$$\frac{1}{2} P e = \frac{T - T'}{2} \frac{W'^2}{3} + \frac{W'^2 T'}{2}$$

Since $\frac{1}{2} P = \frac{T + T'}{2} W'$, then $e = \frac{2 W'^2}{3 P} \left(\frac{1}{2} T + T' \right)$

Let $W' = 2.66$ inches; $P = 60$. and $T = 15$ tons. $\therefore T' = 7.56$ tons and $e = 1.2$ inches.

This last value is the one used in working out Figs. 20 and 21, and is found to vary but little from that for $W' = 4.84$ inches.

Here it is to be remembered that in the investigation by which $W' = 4.84$ inches was determined, the sides of the head were not supposed to approach each other.

This fact, together with other considerations, makes it not difficult to account for the disagreement between the value of W' actually required

and that given by the formula. When tension is first impressed upon the bar, tensile stress exists at A'' , Figs. 20, 21, of comparatively high intensity, while at A , either compression exists or tension of an intensity much less than at A'' . As the amount of tension on the bar increases, the tensile intensity of the stress at A'' also becomes greater, and the same effect may take place at A , though it would often not; but whatever else happens, that at A'' will be largely in excess of that at A , if the metal does not stretch in that section. Really, however, all material is elastic, and wrought iron has an extraordinary capacity for stretching near its point of ultimate resistance. Now, since the intensity of the tensile stress at A'' much exceeds that at A , metal will stretch very much section more at the former point than it will at the latter, and consequently the AA'' will move towards the centre C , throwing the centre of stress further away from A'' . This operation will of course, distribute the total stress more nearly uniformly over the section, decreasing the intensity at A'' and increasing that at A . In this, a simple absence of rigidity, is probably to be found almost the entire cause of the discrepancies between the formulas and the experiments. The movement at AA'' towards the axis of the bar will continue until arrested by the pin; but that portion in the vicinity of G has nothing to interfere with it, and will move on as the total tension is increased, causing the pear-shaped hole which has so often been noticed when bars have been subjected to severe tension.

If the operation explained above, would take place in a head proportioned by the formula, the effect would be much more marked in the case of $W' = \frac{2}{3} W$, which is about the ordinary practice. The tendency towards a high intensity of tension at A'' , and the consequent stretching at that point, are found in practice not to be serious matters, for the rule, $W' = \frac{2}{3} W$ gives results perfectly safe. There is only the tension in the body of the bar to pull apart the head on the line AA'' , Figs. 20, 21, and if the metal in the vicinity of A'' should be strained beyond the elastic limit by a heavy passing load, the experiments of Prof. Thurston and others show that its ultimate strength would not be impaired for the next load, provided the previous strain was not too near the ultimate limit. If in any case, the very slight lengthening of the bar, resulting from the stretching of the material, should be especially objectionable, it would be better to make $W' = W$.

The point of contra-flexure for the part $AA''B$ is found by placing Eqs. 95, 98, or 102—as the case may be—equal to zero. It appears from Figs. 20, 21 that for these particular cases the largest pin requires the most metal. This is for the same reason, that the one of two cylinders which has the largest diameter, will require the thickest sides when both are subjected to the same intensity of internal pressure. The intensity of pressure against the inside of the head is essentially limited by the ultimate resistance of the material.

It might at first seem hazardous, to allow any of the metal in front of the pin to be subjected to its ultimate compressive resistance; the fact is, however, of little or no importance. There is only a mere film so subjected, for the radial compression, q , decreases very rapidly as the interior surface of the link head is receded from. Its value may be found for any point by Eq. 3, after substituting the values given immediately before *Example IV*.

An inspection of Figs. 20, 21 show that the points D should receive special attention in designing a head. At an angular distance of about 35° on each side of the axis of the bar, the width of the metal should be about seven-eighths that of the body of the link, for the larger pin and three-quarters, for the smaller.

Another important point is at H . With a 3.5-inch pin at an angular distance of 40° from the axis, the width should be at least seven-eighths W , and for a 3-inch pin, only a little less. The movement of the centre of stress in AA'' exposes the part GH to a somewhat greater bending moment.

The space, which the pin is buried in the head, is given by Eq. 23, and the maximum compression of any of the radial elements of the pin, by Eq. 34. These formulæ apply, of course, only within the elastic limit. The sum of these two quantities is the increase of distance between the centres of the pins caused by compression alone.

The investigation of the shearing or tangential stress in the material of the head in front of the pin, leads to an interesting point in the comparison of perfect and imperfect fitting pins, and its intensities at a few points will be found.

The shearing stresses at any point included within radial lines drawn through the extremities of the arc of contact have two different sources. The first is that other component which exists with l'' , to make up the force acting directly on the radial section passing through the point.

This stress is uniformly distributed, hence its intensity is constant. Calling it, (the intensity) S' , there results :

$$S' = \nu'' \cotan. (\alpha_2 - \Theta) \dots\dots\dots (122)$$

The other is the shearing stress which results from—or is in connection with—the internal compression and tension q and f , and may act on any but two sections.

The quantities q and f are given by Eqs. 2 and 3 after making the proper substitutions :

$$f = \frac{q_0 r_1'^2}{R'^2 - r_1'^2} \left(\frac{R'^2 + r'^2}{r'^2} \right) \text{ and } q = \frac{q_0 r_1'^2}{R'^2 - r_1'^2} \left(\frac{R'^2 - r'^2}{r'^2} \right);$$

in which r'_1 and R' are the interior and exterior radii of the imaginary head, and r' any intermediate distance.

If r_1 , R and r are the corresponding actual quantities, there must be substituted in the formulæ for q and f :— $r'_1 = r_1 A$, $R' = R - (1 - A) r_1$ and $r' = r - (1 - A) r_1$. Eqs. 24, 25 and 26 gives the value of A .

The only formulæ required in this case are those given in Art. 112 of Rankine's "Applied Mechanics." Let S be the intensity of this shearing stress ; its general value is given in Eq. 4 of that article, but the one following, (Eq. 5), gives the maximum value, and it only will here be applied ; hence maximum value of

$$S = Sm = \frac{q + f}{2} \dots\dots\dots (123).$$

The maximum value of S will exist at the point B' , Figs. 23, 24, on the axis of the bar when t'' and therefore S' , equals zero.

Eqs. 122 and 123 will be applied in *Example V*. At the point B' , for which $r = r'_1 = 1.5$ inches, $f_0 = 25.75$ tons.

Deducting, $t = 13.2$ tons, for intensity of compressive bending stress, there results : $f = 12.55$ tons ; then, since $q = 25$ tons,

$$Sm = \frac{f + q}{2} = 18.8 \text{ tons} \dots\dots\dots (125).$$

Eq. 125 gives the maximum value of S , and neither it nor any other value exists in the same plane with t'' , in reality ; when the latter occurs, the former is zero.

Using data from the same example in Eq. 122, there results from the different values of Θ : for $\Theta = 41^\circ 15'$, $t'' = 3$ and $S = 4.4$ tons ; for $\Theta = 20^\circ$, $t'' = 8$ and $S = 5.45$ tons ; and for $\Theta = 0$, $t'' = 12.5$ and $S = 3.2$ tons.

It is seen from these results, that if other conditions of safety are complied with, rupture by the shearing stress S is abundantly provided

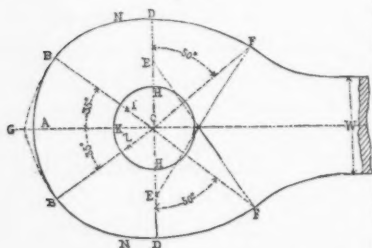
against, since the shearing resistance of wrought iron per unit of area is about 23 or 24 tons. The same cannot quite be said of S . The value of q is preserved essentially constant by the ultimate limit of compressive resistance, so that as the tension in the bar is increased, the pressure around the pin is more nearly uniformly distributed and the condition of fitting perfectly is approached. Thus, the value of f , in Eq. 123, might be increased to such an extent that S would reach its ultimate limit before T , the tensile intensity on the outside of the head.

Ordinarily, however, if all other matters are attended to, that of the shearing stress need cause little anxiety.

The collapsing tendency of the sides of the head on the pin, from flexure, has not been investigated, since its consideration would involve such a degree of uncertainty that the results would be valueless or little better; besides this neglect does not, probably, seriously affect the results obtained.

It is not readily predicted from the preceding results how an eye bar head would fail in an experiment. However, it is probable that the effects of compression and shearing would predominate in the resultant stress at the point of rupture.

Fig. 26.



The method of constructing a link head shown in Fig. 26, gives a proper disposition of material with a very simple construction. The quantity between B and D is slightly in excess of that required, on account of the point of contrary flexure. The figure shows a 4-inch bar with a 3.5-inch pin. A smaller

pin would not require quite so much metal, and a larger one would require a little more, but the rule gives dimensions a little in excess of those required by the formulæ; besides, a greater ratio between diameter of pin and width of bar is seldom used.

Let r = radius of pin, and W = width of bar, then;

$$BC = AC = r + \frac{7}{8}W, DH = \frac{2}{3}W \text{ and } ED = EF = 2r + W.$$

DF is described with ED until $DCF = 50^\circ$, and BAB is described with BC until $BCA = 35^\circ$. BN is drawn from L as a centre in such a manner as to be at the same time tangent to DN and AB . DN is a

There is now to be found the sum of all the axial components MT ; call this sum F' . By reference to Fig. 27, it is seen that the general value for $d(F')$ is:—

$$d(F') = \varphi r_2 q d\Theta', \sin. (\alpha_2 - \Theta') - \\ \varphi r_2 \operatorname{cosec}. (\gamma + \Theta) \cos. (\gamma + \Theta) q \sin. (\gamma + \Theta') d\Theta'.$$

Taking Θ between β and α_2 , and remembering that there will be two parts of $d(F')$:

$$F' = \int d(F') = \\ \varphi r_2 q_0 \left\{ \frac{\sin. (\alpha_2 - \beta - \sin. \alpha_2 + \cotan. (\gamma + \Theta) [\sin. \gamma - \sin. (\gamma + \beta)]}{\beta} + \right. \\ \left. \cos. (\alpha_2 - \Theta) + \frac{\cos.^2 (\gamma + \Theta)}{\sin. (\gamma + \Theta)} \right\} \dots (130).$$

$$\text{If } \Theta = \alpha_2; F' = \varphi r_2 q_0 \left\{ 1 + \frac{\sin. (\alpha_2 - \beta) - \sin. \alpha_2}{\beta} \right\} \dots (131).$$

When Θ is less than β :

$$F' = \varphi r_2 q_0 \frac{1}{\beta} \left\{ \Theta \cos. (\alpha_2 - \Theta) + \sin. (\alpha_2 - \Theta) - \sin. \alpha_2 + \right. \\ \left. \cotan. (\gamma + \Theta) \left(\Theta \cos. (\gamma + \Theta) - \sin. (\gamma + \Theta) + \sin. \gamma \right) \right\} \dots (132)$$

If q is not constant at any part of the surface of contact, Eq. 132 gives the value of F' by simply changing β to α_2 . Making $\Theta = \alpha_2$, under this supposition:

$$F' = \varphi r_2 q_0 \left\{ \frac{\alpha_2 - \sin. \alpha_2}{\alpha_2} \right\} \dots (133)$$

Each of the small forces BT , has a moment about the centre of any section. The lever arm of this force is:

$$\frac{R + r_2}{2} \cos. (\Theta - \Theta') - r_2.$$

Therefore the differential of the total moment is:

$$d(M) = \varphi r_2 \frac{R + r_2}{2} q \cos. (\Theta - \Theta') d\Theta' - \varphi r_2^2 q d\Theta'.$$

First let Θ be taken between α_2 and β :

$$M_1 = \varphi r_2 \frac{R + r_2}{2} q_0 \left\{ \frac{\cos. (\Theta - \beta) - \cos. \Theta}{\beta} \right\} - \\ \varphi r_2^2 q_0 \left(\Theta - \frac{\beta}{2} \right) \dots (134).$$

When Θ is less than β :—

$$M_1 = \varphi r_2 \frac{R + r_2}{2} q_0 \frac{1 - \cos. \Theta}{\beta} - \varphi r_2^2 q_0 \frac{\Theta^2}{2\beta} \dots (135).$$

If q is not constant at any part of the surface of contact, β is to be replaced by α_2 in Eq. 135. The effect of friction is allowed for, by adding some one or more of the preceding equations, or subtracting as the case may be, to the second members of those given for the case without friction.

The values of N' in Eqs. 128 and 129 are to be subtracted from N of Eqs. 24, 25, and 26 in order to get the new value of the normal component. Denoting this new value by N_1 , there results:— $N_1 = N - N'$. N and N' are to be taken, of course, from equations applying to the same circumstances.

Since a part of the force P is used in developing the force of friction, the value of F' , Eq. 133, must be added to Eq. 39. That equation of condition will then stand:—

$$\frac{1}{2} P = \frac{q_0 r_2}{\alpha_2} (1 - \cos. \alpha_2 + \varphi \alpha_2 - \varphi \sin. \alpha_2) \dots (136).$$

The quantity q_0 can then be determined in the same manner as before.

The moment M_1 , Eqs. 134 and 135, has a sign contrary to that of M' in Eqs. 47, 48, 49 and 50. Therefore if for M' there is substituted:

$$M' - M_1 = M'_1 \dots \dots \dots (137)$$

the moment of friction will be allowed for.

The next equations effected are 58, 59, 60, 61 and 62, in which F must be replaced by $F + F'$, so that:—

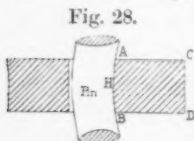
$$P' = \frac{1}{2} P - F - F' \dots \dots \dots (138).$$

Finally, F' , from Eq. 130, is to be added to the second member of Eq. 80. In short, F' is to be taken from Eqs. 130 to 133 and added to the second members of Eqs. 80 to 83, according as any pair of formulae are devised for the same case. After the substitutions which have been indicated are made, the proceedings are the same as for the case with no friction.

The value of φ is exceedingly uncertain, if known at all, for the circumstances under which it here exists. This fact, coupled with the doubt in reference to the amount of frictional action, takes away all real value of the equations. The omission of the effect of friction, however, is an error on the side of safety, and so small a one that it is more advisable to commit it than to take the risk of the commission of another on the side of danger.

The method of dealing with a thickened head has already been indicated.

It has been assumed in the whole of the previous investigation that the pin retains its straight centre line under stress. This, it is evident, is not strictly true, but a little consideration will show that the effect of the bent pin is of scarcely any importance. Fig. 28 is a longitudinal section of the head, with the pin in position,



tudinal section of the head, with the pin in position, under stress. Before the limit of elasticity is reached, the intensity of the stress at *A* and *B* will considerably exceed that at *H*, the middle point; and it will vary between those points. For

a differential of AH , the intensity of the common normal pressure may be taken as constant. Now, the head may be supposed to be divided into a great number, *i. e.*, $\frac{1}{d(AH)}$, of parallel layers with different but constant intensities of normal pressure, each with a thickness equal to a differential of AH , and each may be discussed as a single head. The state of strain in $ABCD$ tends toward uniformity as AB is departed from, and it is probable that that state is attained not far from that line. After the ultimate limit of compressive resistance is reached along AB , the only difference between the conditions of stress at *A* and *H*, results from the difference in radii of pin-hole at the two points, and that is essentially inappreciable. Any state between the limits described, will partake of the nature of both, so that in any case, the effect of the pin curved by bending is of little account.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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CLXI.

CONNECTED-ARC MARINE BOILERS, A DEMONSTRATION OF THE PRINCIPLES OF THEIR CONSTRUCTION.

A Paper by CHARLES E. EMERY, Member of the Society.

READ DECEMBER 20TH, 1876.

The tendency, of late, has been to increase the steam pressures carried in marine boilers, for which reason boilers of circular section, which have sufficient strength with little internal bracing, are preferred to those of rectangular shape requiring a large number of braces. In general, when circular boilers are used, there is considerable space necessarily wasted in the spandrels, which in the rectangular system, can be made part of the internal capacity; and when the height is limited, so large a number of circular boilers are required in a naval steamer that it becomes difficult to keep the feed water distributed uniformly. To utilize the strength of the circular form of boiler and permit in great measure the changes of shape and size with limited height which are possible with rectangular boilers, the writer some years since developed a system in which the boiler shells, in section, were composed of a series of connected circular arcs, supported by other arcs either directly or through ties or struts connecting the junctions on one side of the boiler with those of similar arcs on the other side.

Four marine boilers involving this principle have now been in use since the year 1874, which, when in process of construction from plans of the writer, excited considerable discussion and many predictions of failure, though the critics could not agree among themselves whether rupture would take place, vertically or horizontally. No calamitous

results have yet followed their use. We have seen in a publication of the Hartford Steam Boiler Insurance Company, a sketch of a very small boiler, somewhat similar in shape, but differing greatly in detail from those above mentioned, which actually exploded, doing such damage as to be made the text of remarks about the impropriety of using untried forms. The sketch was sufficiently complete to show that the design had not been properly considered, and it is more than probable that the boiler had been made without any conception of the nature of the strains developed.

It is believed that the conditions of equilibrium, of structures with sectional outline composed of a series of connected arcs, substantially as shown in Figs. 3 and 8,* may be as definitely ascertained as if such sections were complete circles. In presenting the subject to the Society, it is considered that it will be of interest to follow the order necessarily pursued by the writer, and investigate first the principles involved in such construction, and then present the manner in which the same have been successfully applied in practice.

The investigation develops some features which are applicable as well to arch bridge construction, and may be the means of bringing out something in that direction.

Strains in structures subjected to fluid pressure are most readily investigated in the manner referred to, by the writer in a discussion on the subject of bridges, at a meeting of the Society, February 16th, 1876; the application of which system to the present case may be briefly explained, as follows:

A body, supposed to be without weight, if subjected to pressure in a fluid, supposed also to be without weight, will remain at rest under equal opposing forces, no matter what the shape of such body may be. For the present purpose, suppose such body to be a right prism, one unit of the scale in height. The lengths of the sides of either base would then, in terms of the superficial unit, represent the magnitude of the forces, measured by the pressures normal to the several faces. If, for instance, Fig. 11 represents the polygonal base of such a prism, the several sides represent the pressures on the faces of the prism or the magnitude of forces acting normal to such sides in the directions of the arrows. The original prism can, however, be divided into lesser prisms by planes crossing the bases at any angle. The new prisms will also be in

* The figures referred to in this Paper, are to be found on plates, opposite page 176: Figs. 1-11 being on Plate XV; Figs. 12 and 13 on Plate XVI, and Figs. 14-17 on Plate XVII.

equilibrium under forces measured by the several sides of the polygon, consequently the forces at the planes of separation are equal and opposite and hold in equilibrium all the forces represented by the remaining sides of the polygon. For instance, a force represented by GF holds in equilibrium forces measured by GA , AE , ED and DF , also those measured by FC , CB and BG . The directions of all forces are either from the interior of the figure outward or the reverse. The result would not be altered were either or all the sides, original polygon curves.

Any combination of forces, then, represented by the sides of a figure, may be resolved into a single force, measured by a line crossing the figure at any desired location and equal to the length of such line multiplied by the pressure per superficial unit.

Hence if Fig. 3 represents a section of a boiler consisting of a series of connected circular arcs— O_1, O_2, O_3 and O_4 , being the centers of the circles, all located in the line xy ,—the vertical strains tending to separate the top of the boiler from the bottom, are measured by xy , and the horizontal strains at XOW by XW . As the strains above and below the line xy , are equal and opposite, we will at first consider the former only. Lines drawn across the figure, parallel to xy , would represent the vertical strains at such locations, which would equal zero for a line tangent to a circle, at W , for instance. At this point then, no vertical strain exists and none can be transmitted as the material is disposed horizontally. All the resolved forces concentrated at such point are horizontal and measured by OW . Similarly, at x there is no horizontal strain and all the strains are vertical and measured by Ox . If then, the quadrant xW , and figure $WOYU$ were detached, and considered as bases of separate prisms under fluid pressure, as above set forth, the horizontal strains concentrated at W , would in each figure be measured by WO , and those in latter at U , by UY ; similarly the vertical strains at U , being the resolved forces received by the arc WU , would be measured by the horizontal component OY . Thus it will be seen, that the vertical and horizontal forces concentrated at the end of an arc (WU for instance), are measured by the sine and cosine of the arc, or what is the same thing, the projection of the arc and of its complement on rectangular co-ordinate axes—the force represented by one projection acting in the direction of the other.

The above method of demonstration not being familiar, though it employs practically a modification of the well known equilibrium polygon, it has been thought proper to make an investigation of the same subject

the ordinary way ; in a foot note.* The result being the same in both cases, we proceed with the analysis as follows :

Since the vertical components of forces cease at co-ordinates passing through the centers of the circles, the vertical strain from the arc, WU , concentrated at U , is measured by OY , and similarly the vertical strain from arc, UW_2 , concentrated also at U , is measured by YO_2 . The total strain at U is then measured by $OY + YO_2 = OO_2$, and may be met by a tie extended from U across to V , a corresponding point in a symmetrical system—a strut being required instead of a tie, if the pressures are on the outside of the structure. Similarly the strains at U_2 and U_3 are measured respectively by the distance, between the centers O_2, O_3 and O_3, O_4 . By inspecting Figs. 4, 5, it will be seen that the vertical strains at the ends of the arcs are always measured by the distance between the centers, though the diameters of the shell may vary. The horizontal strains at either junction in these figures are, it will be seen, measured by the half length of the ties, UY, U_2Y_2 , (the projections of the complements of the arc). It is evident that either of these forms may be extended laterally, indefinitely.

It is not necessary, however, that the centers be in the same line. In Fig. 6, an arc UU_2 is shown connected to two arcs of greater radius. The vertical strains on ties in this system are, as before, measured by the distances between centers. The horizontal strain of the arc UW , with larger radius, is measured by UY , while that of arc UU_2 is measured by UY_2 , less than UY , hence the arc of larger radius (under internal pressure), tends to straighten the arc of less radius by a force measured

* In Fig. 1, let FE represent an arc of a circle, O being the centre. Let FA, AB, BC , &c., represent sides of a regular polygon inscribed in the circle, and OF, OA, OB , &c., radii, meeting the circumference at the ends of the several sides of the polygon. Draw GF and GB parallel to AB and AF , thus forming a parallelogram, $ABGF$.

A fluid exerts pressure equally in all directions, which pressure acts normally to the inclosing walls of the vessel, or radially upon any parts of the walls that are of circular section. (This is modified by the weight of the fluid, but in boilers constructed for high-pressure steam, the modification of pressure due to the weight of the water is so small, comparatively, that it need not be considered here.)

If, in the parallelogram $ABGF$, GA be taken to represent the intensity of a radial force acting in the direction of O towards A , such force will be held in equilibrium by two forces, represented in intensity and direction by the two sides of the polygon AB and AF , and acting from A towards B and F . Similarly constructing the parallelogram of forces, $BCHA$, the radial force, $BH=AG$, will be held in equilibrium by two forces, BA and BC , acting from B toward A and C respectively. But the force, AB , in the first parallelogram, $ABGF$, is equal in intensity and opposed in direction, to the force BA , in parallelogram $BCHA$. In like manner, it may be shown that the radial forces at any other point— C , for instance—will be resolved into two forces acting in the direction of sides of the polygon, half of which force will on either side balance and hold in equilibrium half the force due to the resolution at the adjoining points, B and D . If, then, the number of sides of the polygon be indefinitely increased, the same resolution of radial forces will take place at each angle, and when the number of sides becomes infinite, and the polygon a circle, the resolution will take place at

by $UY - UY_5 = FY_5$, which may be prevented by a tie UU_2 connecting the ends of the arc of less radius.

In Fig 7, an arc UU_2 is shown between two of less radius. The horizontal strains, of the arc of larger radius, being greater than those of the former by an amount FY_5 , the outer ones will fail of themselves to furnish the junction sufficient support, and for internal pressure a strut must be placed from U to U_2 , with section proportioned to sustain a strain measured by FY_5 .

In Fig. 8, the system shown in Fig. 3 is still further extended, without, however, varying the nature of the strains, as will be found by applying the previous considerations to the arcs both vertically and horizontally.

It will be evident from the above that modifications may be made to suit any location, while retaining the strength of circular sections. The simple form shown in Fig. 3 is generally best applicable, but that shown in Fig. 8 can well be used in place of rectangular boilers, thereby concentrating the braces, making strains on the same more definite, reducing the surface exposed to corrosion and giving more room for access. Two curious forms illustrate the adaptability of the system.

Fig 9 represents an approximation to a triangle, made by combining three arcs of circles. The strength of shells should be proportioned to radius CA . Under pressure, each arc would tend to assume greater convexity, which can be prevented by struts connecting with the junctions. Were the arc AB considered alone, the strut connecting A and B would be proportioned to CD , but as this arc is attached to arc AC , the pull of every possible point of the circumference. The resolved forces will act in the direction of the tangents at those points, and—since, as above shown, half the resolved force in one direction along the circumference balances an equal force in the opposite direction from an adjacent point on the other side—it follows, in general, that :

The resolved tangential forces do not increase as the length of the arc is increased, but that the strains are the same at all points of a circular arc, and independent of the length of the same.

Referring to Fig. 2, in which the arc MM_1 is a quadrant and MM_1P , a semicircle, MP being the diameter, if we represent the resolved tangential force at any point, M_2 for instance, by a tangent M_2V , made equal to the radius M_2O , the strain at such point, per unit of length of cylinder, will be equal to the radius multiplied by the pressure per square unit, or, in boiler construction, the strain per inch in length of shell will equal the radius in inches multiplied by the pressure per square inch. But we may resolve the force, represented by the tangent M_2V , into two forces, represented by VW , acting parallel to the diameter MP , and WM_2 at right angles to the same, which lines will then represent the intensities and directions of two forces which will balance the tangential force at that point M_2 . But the triangles M_2VW and M_2ON_2 are similar, the sides of each being severally at right angles to those of the other, so the radius of the arc MM_1 or OM_1 in triangle M_2OV_2 may be taken to represent the intensity of the resolved tangential force at that point, acting in the direction of the tangent, when M_2N_2 (equals sine of arc MM_2) and ON_2 (cosine of same arc) will represent the intensity of the orthogonal forces required to balance the tangential force at the point M_2 , each of said orthogonal forces acting, however, in the direction of the other.

which, in direction EB is measured by AE ; resolving the latter, we have a force EF , equal DG , assisting to maintain the shape of arc AB , so the strut AB should be proportioned to $CD - DG = CG$.

In Fig. 10, are shown four connected arcs with their convex sides turned toward an enclosed space approximating a rectangle in shape. In this case, for internal pressure, the ends must be connected by external ties. That on side AD , should be proportioned to $FG + AL$; the former being a measure of the section required to keep the arc in shape, the latter representing the vertical lift at A , of the arc AKB , which in this case does not, as in previous cases, assist to keep the arc AD in shape. This particular form would therefore not be economical in material, but it would have the advantage in some locations of requiring no interior braces. It is a curious fact, that if a convex arc AJB , with center at O , were substituted for AKB , it would need no braces, the vertical pull as before being transferred to tie AD , and the horizontal pull, measured by AG , being balanced by that of half the arc AD , measured also by AG . The action would be the same if AG were a flat surface, trussed to maintain its shape.

In general, we may conclude that the outline of a boiler may be made of any possible combination of circular arcs, all of which will retain their shape under pressure, if the ends of same are supported by each other directly or through ties, and struts, arranged and proportioned on the principles herein-before developed.

Fig. 12 is a front view, partly in section, of a boiler built in 1873-4 for the U. S. Revenue Steamer, *Rush*, which embodies the connected arc system of construction above analyzed. Fig. 14 represents a longitudinal central section through same, and Fig. 13 a top view of the steam chimney. As shown, the shell is made with two arcs, each

If we consider MP and M_2O rectangular co-ordinate axes, passing through the center O of the circle, then the two forces required to hold in equilibrium the end of any arc forming part of the quadrant MM_2 will be measured respectively by the projections of the arc and of its complement upon the co-ordinate axes, and equal the length of such projections multiplied by the pressure per superficial unit. For instance, the horizontal component required to balance the tangential force at M_2 is measured by $M_2N_2 = \sin$. $M_2OM = OT_2$, or the projection of the arc M_2M on axis M_2O , and the strain equals OT_2 multiplied by the pressure per superficial unit. The vertical component is similarly measured by $N_2O = \cos$. M_2OM , or the projection of the arc M_2M_2 , which is the complement of arc MM_2 , on the axis MP ; and the strain equals N_2O multiplied by the pressure per superficial unit.

It follows then, that the strains concentrated at any point of a circular shell subjected to fluid pressure may be held in equilibrium by a force measured by the radius and acting at a tangent to that point, or by orthogonal forces measured by the projections on co-ordinate axes of the two arcs included between such point and the axes; the actual value of strains equalling in each case, in boiler construction, the length of the radius, or of the projections named, in inches, multiplied by the pressure per square inch.

greater than a semicircle, joined by flanging the ends of the sheets inwardly, and riveting same together through a central plate *b*, as shown more clearly in Fig. 15. It not being possible to put the rivets exactly at junction of tangents, a covering plate, *j*, is applied to stiffen the angle, and the triangular enclosed space connected with interior of boiler by means of slits, *p, p*, which are only one inch wide, and separated about 6 inches, so that they do not weaken the shell as much as the rivets. In a boiler of same kind, built for the U. S. Revenue Steamer *Boutwell*, where the angle of tangents was more obtuse, the shells were connected by a spandrel piece, *F*, Fig 16, and the vertical strain taken by a series of large bolts, *b*, the junction not being exposed to the fire.*

The intermediate tie-plate *b*, is perforated with a number of vertical openings. Fig. 14, which maintain communication between the two shells. Further communication is also obtained through back and front water spaces, *G* and *J*. From the front and rear water spaces, water legs *H, H*, are run longitudinally under the boiler, and connected to same by exterior flanges. The legs and shells are connected by means of slits about one inch wide, spaced 6 inches apart, as shown in plan in Fig. 17, which reduce the section of shell less than the rivet holes. Between the water walls, at the front, are grates, and at the rear, combustion chambers, the latter being closed at bottom with fire brick, which can be readily removed to effect repairs.

The bottom of the boiler shells are directly exposed to the fire, and the products of combustion pass through combustion chambers to a back connection *L*, then through tubes *T*, to a front connection *M*; thence through two flues *N, N*, in steam chimney to the smoke pipe. The steam chimney is made double, like the main boiler shell. In two vessels, the *Rush* and *Dexter*, built for high pressure steam, the steam chimneys are separate structures connected to boiler by tubes, *O, O*, as shown. Such connections do not materially weaken the shells, and for high pressure steam are preferred to building the chimney on the boiler, and cutting out the full area. On the two vessels named, the front connections, *M*, were built without water walls, as shown. On a third boat, the *Dallas*, using low pressure steam, the front connection was built in and the steam chimney built on the boiler. The tubes are kept above the bottom of boiler shells sufficiently to permit access directly to crown sheets, through a man-hole *P*, in each shell. At rear of boiler, opposite

* The governing dimensions of this boiler will be found in the Paper on "Compound and Non-Compound Engines, Steam Jackets, &c.," by the writer, Vol. III, page 368.

man-holes, water-tubes, *Q, Q*, connect the interior of shells with rear water space. The stability of the flues *N, N*, in the steam chimney is assured by making same in short sections, with ends of latter flanged outward and riveted together through calking rings. The main longitudinal stays, *R, R*, pass between pairs of heavy angle irons, riveted to each head, and have inside nuts bearing on combined sockets and washers catching over flanges of angle iron. The outer nuts bear on large washers. The bolts are jointed centrally so that they may be readily removed when corroded.

The hulls of the revenue steamers are made as small as possible, to enable them to carry powerful machinery with necessary fuel and stores; consequently, space is an important object. By adopting the plan above described, boilers of necessary strength for a steam pressure of 80 pounds were made sufficiently low to enable the space in the deck houses, above, to be utilized.

In regard to the strength of these boilers we never felt any solicitude, as the strains were readily calculated, as set forth, and there was no difficulty in putting the material where it was needed. There were, however, several questions as to durability. The limited openings from water walls to shell were criticised by some, but a slight consideration showed that the communicating spaces were ample for the amount of heating surface in the legs. It needed investigation to determine what thickness of metal would stand without burning over the fire, also how the longitudinal joints, where the legs connect with boiler, would withstand the heat. The ample water space above the crown sheets, making the latter readily accessible for cleaning, has kept them intact, although on one of the vessels the metal is $\frac{1}{16}$ -inch thick. As to the longitudinal joints, it was found that legs had been before applied similarly to round shells, without giving trouble.

The results of practical trials have not developed any weaknesses, and no leaks have occurred due to any of the peculiar features of construction. The lower leg seams occasionally have opened slightly, after using cold water, the same as in other boilers. The boiler of the *Boutwell*, above referred to, had a rectangular fire-box, with attached double-arc shells, which received the flues and tubes for both the direct and return products of combustion. In plans submitted to the U. S. Navy, the grates, instead of being placed under the shells, are located in circular internal furnaces, which, everything considered, is probably better, though not quite so economical in space, as the arrangement shown.

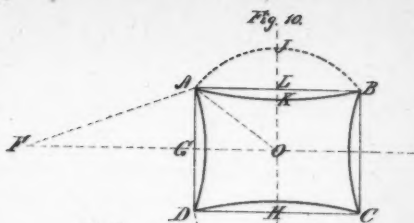
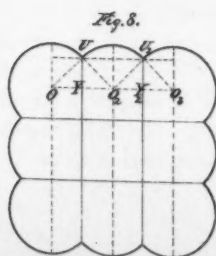
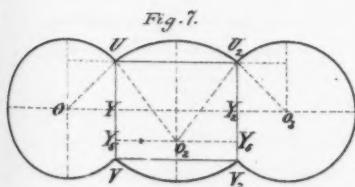
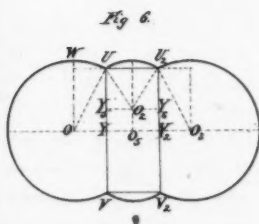
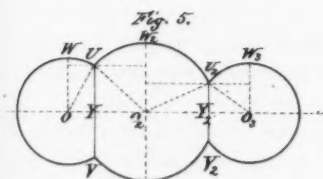
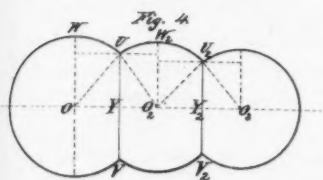
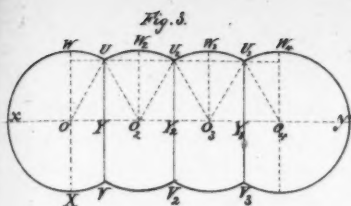
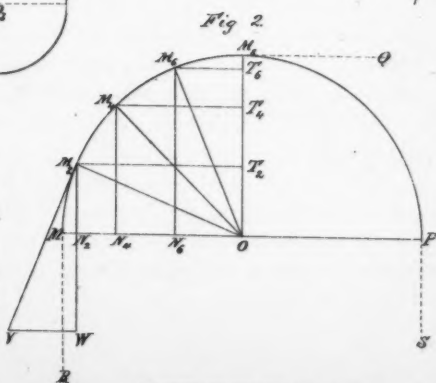
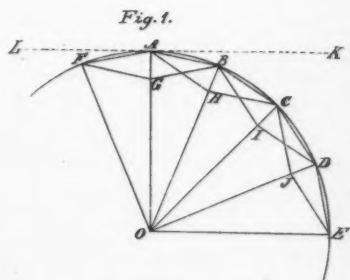
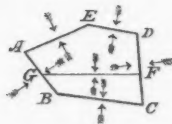
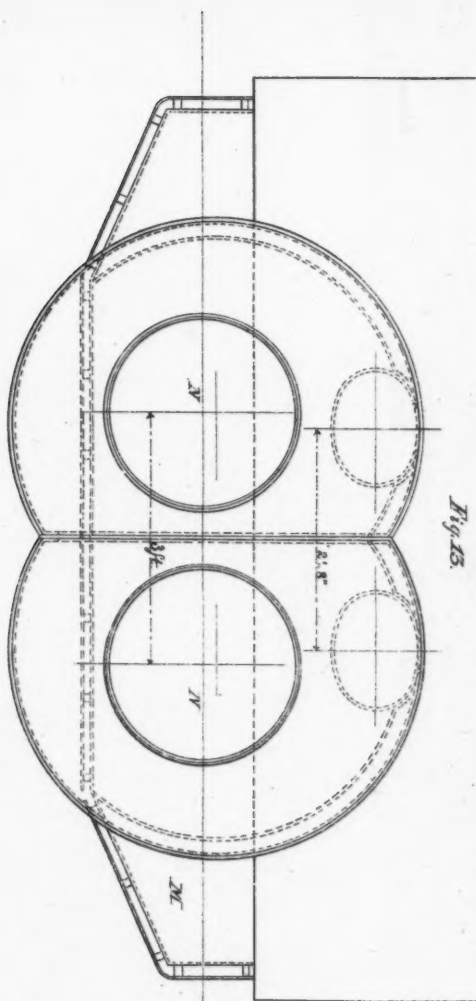


Fig. 11.







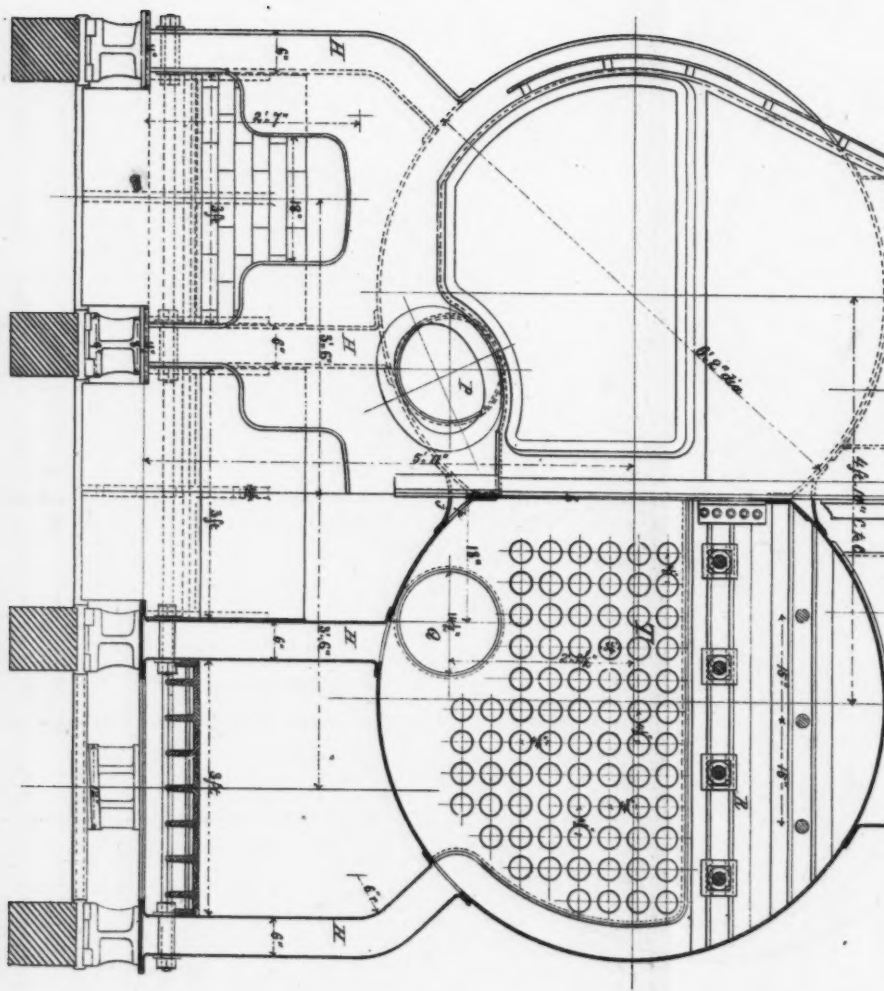
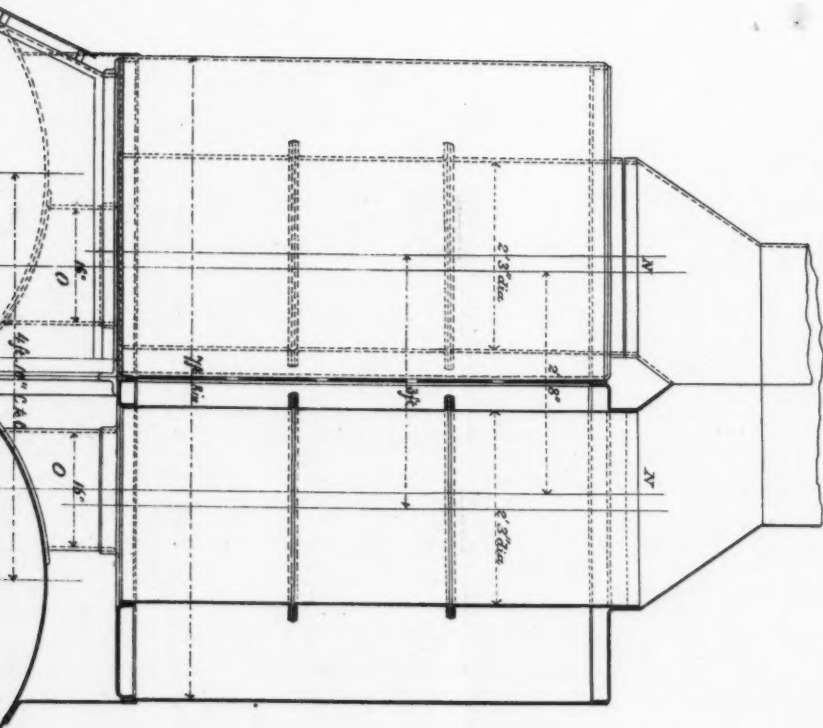
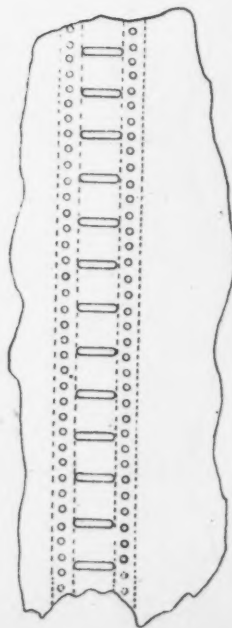
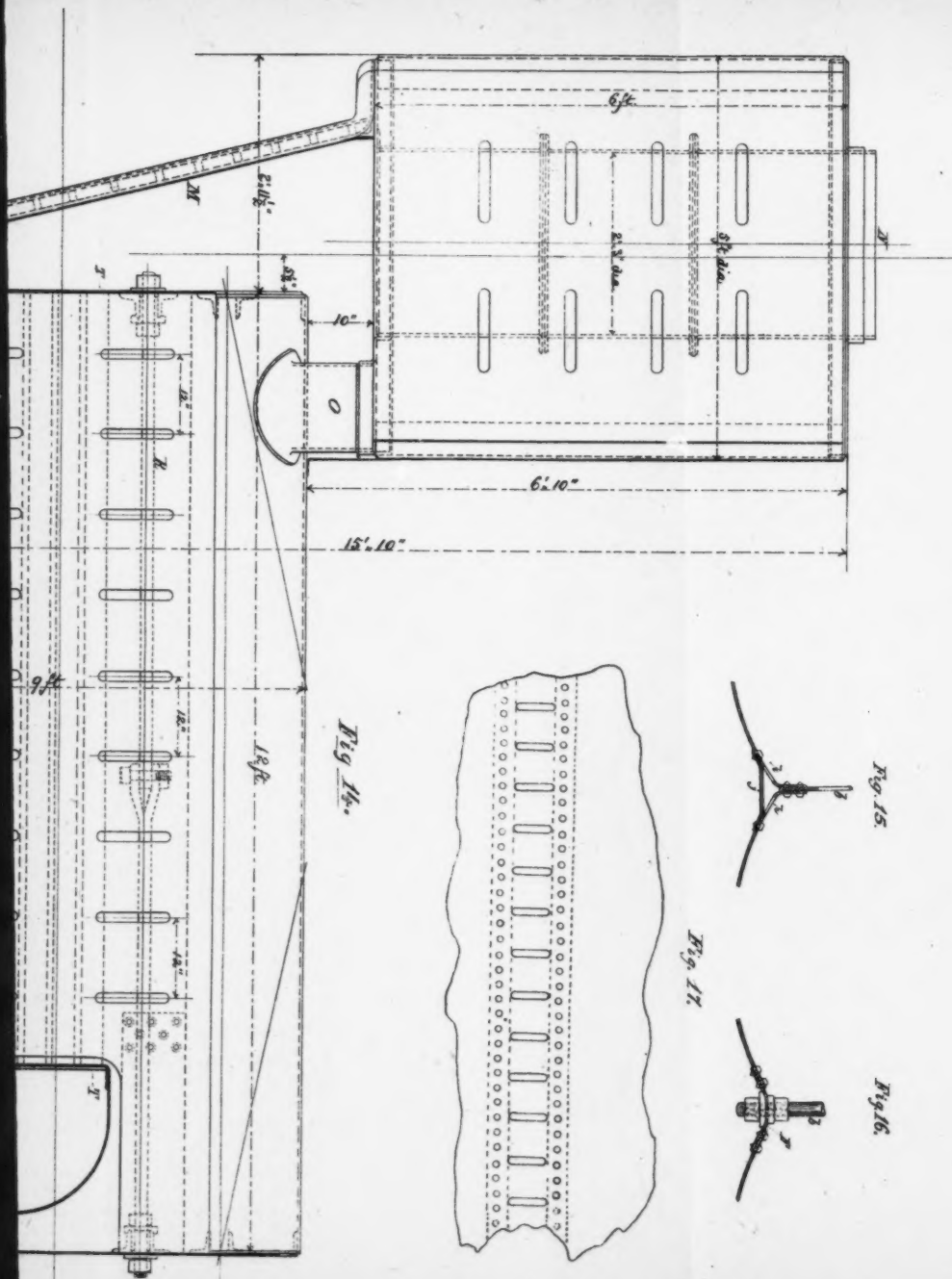


Fig. 12.







ON THE SIMULTANEOUS
IGNITION OF THOUSANDS OF MINES,
AND THE MOST ADVANTAGEOUS GROUPING OF FUSES.

A Paper by JULIUS H. STRIEDINGER, C. E., Member of the Society.

READ APRIL 4TH, 1877.

THE SIMULTANEOUS FIRING OF THOUSANDS OF MINES MAY BE EFFECTED :

First.—By relying upon the laws of transmission of detonation, (Trauzl's method).

Second.—By the application of a "multiple circuit closer" to our electrical blasting apparatus, (the writer's method).

Third.—By a combination of both methods, (Gen. Newton's method).

I.—ON THE TRANSMISSION OF DETONATION.—Capt. Trauzl and Lieut. von Treumfeld, Imperial Austrian Engineers, and Prof. F. A. Abel, have thoroughly investigated the first proposition referring to blasting on land.

Feeling the want of more precise knowledge on the subject of transmission of detonation when employing the agency of water, repeated experiments were made by the writer under the direction of Gen. Newton, U. S. Engineers, until the desired information had been obtained.

The following practical rules are the result of these important investigations :—

A.—TRANSMISSION OF DETONATION ON LAND.

Assuming, a proper fuse being employed for detonating the initial charge, the transmission of detonation to hard frozen and soft dynamite by means of wrought iron pipes (English gas pipes) of $1\frac{1}{4}$ inch inside diameter, and $\frac{1}{8}$ inch thickness of metal is possible, without any apparent reduction in the force of the blasting power of the high explosive :

1°.—*If the pipes and charges are fastened together in such a manner as to withstand the tendency they have to separate, due to the lateral transmission of the force developed at the instant of detonation.*

2°.—*If the dynamite charges are open towards the initial cartridge, a reliable transmission of detonation is produced with certainty ; (a), by means of straight pipes, if the charges are 12 feet apart (observed maximum*

distances 26 feet, and in one case 36 feet) ; (b), by means of bent pipes of a radius of curvature not less than 2 feet, when the charges are 6 feet apart ; (c), by combining straight and curved pipes, provided charges of $1\frac{1}{2}$ pounds of dynamite are placed at every 12 feet in the straight parts and at every 6 feet in the curved parts of the system, also at each sharp bend or cross ; (d), coupling does not interfere with the transmission of detonation by these pipes so long as the coupled pipe ends are not more than $2\frac{1}{4}$ inches apart.

B.—TRANSMISSION OF DETONATION IN WATER.

Here, tubing is unnecessary, the agency of water being sufficient to transmit the detonation, under the supposition, as before, that the proper fuse has been used for detonating the initial charge, then *a sure transmission of detonation will happen*, if the following requirements be complied with :—

1°.—*Should the one half pound charges of dynamite or rend-rock be encased in rubber gun cotton bags, forming cartridges 2 inches in thickness, or in cylindrical paper shells of 2 inches in diameter ; these cartridges to be—(a), 12 feet or less apart ; (b), submerged not less than 6 feet.*

2°.—*If the one pound charges of dynamite or rend-rock are encased in brass cans 2 inches in diameter and 6 inches in height, these cartridges to be—(a), not more than 5 feet apart ; (b), submerged not less than 6 feet.*

The dynamite mentioned above is supposed to have the same composition as that used in the writer's experiments, being dynamite No. 1, consisting of 75 per cent. of nitro-glycerine and 25 per cent. of kieselguhr.

The rend-rock to consist of :

33.4 parts of nitro-glycerine,	} by weight.
2.4 " " charcoal,	
6.7 " " sulphur,	
52.8 " " saltpetre,	
2. " " rosin,	
2.7 " " pulp,	

Since the rate of detonation of spaced dynamite masses is (according to Prof. Abel's measurements) 6 239 feet per second, it is evident, that by detonating at the same moment a comparatively small number of initial charges advantageously placed, thousands of mines connected with these "lightning matches" may be simultaneously fired for all practical purposes.

In connection with submarine blasts, where the initial and intermediate charges are placed so as to form projecting parts of the mines, this

manner of explosion seems to deserve special commendation, on account of its simplicity and cheapness. It certainly offers an additional precautionary measure against misfire in blasting by means of electricity.

II.—ON THE APPLICATION OF A MULTIPLE CIRCUIT CLOSER TO OUR ELECTRICAL BLASTING APPARATUS, (the writer's method.) Until recently the method employed for the simultaneous firing of mines by means of electricity, consisted in placing the electrical fuses into the charges, uniting the ends of the fuse and lead-wires in groups or single series, bringing the lead-wires to the battery placed at a safe distance, and then touching off the blast.

This manner of ignition necessitated :

- 1°. A large and expensive battery as source of electricity ;
- 2°. Long and heavy lead-wires as electrical conductors ;
- 3°. Complicated wire joints.

Moreover, it did not offer much probability of success, a break in the single circuit formed by the lead-wires being sufficient to produce a misfire of *all* the fuses.

The interposition of the multiple circuit closer overcomes these defects. When placed and used in conjunction with the batteries close to the mines, but short lead-wires are required, and hence smaller batteries will suffice. Enabling the simultaneous firing of all the charges however divided up in sets of independent groups and batteries, the circuit closer offers in addition the enormous advantage of simpler wire joints and greater security against misfire, since a fault in one of the sets—being here entirely localized—does not affect the other groups.

This apparatus, as employed for firing the final blast at Hallet's Point, consisted of two wooden plates, the one fixed, the other movable vertically. These plates were enclosed by a wooden frame, which supported the first plate, and by means of grooved standards or guides, a free vertical movement of the movable plate, placed immediately over the fixed plate, was obtained. The latter was provided with 24 brass cups filled with mercury ; the brass stems of these cups projected beneath the table, where they ended in screw cups for wire connections. The movable plate carried 24 brass pins passing through it vertically above and corresponding to the 24 mercury cups. The square ends of the pins formed screw cups at the upper side of the plate. This plate was supported by a suspension cord, into which a small cartridge was introduced, whose explosion by an independent battery severed the cord, thus allowing the pins to simultaneously enter the mercury cups, and con-

sequently to close all the open circuits in case the wire connections (viz. : from positive pole of battery to fuses, from fuses to mercury cup, from screw cup of pin to negative pole of battery) of each of the 23 large groups were previously completed.

Of the 24 sets of mercury cups and brass pins of the multiple circuit closer but 23 were needed for the great blast, the other set was used by Gen. H. L. Abbot, U. S. Engineers, for his investigations on the velocity of the transmission of the shock produced. The circuit closer was constructed of well-seasoned wood; when placed in the battery house, its three legs were insulated, by being put into boxes filled with molten sulphur. Rubber tubing and rubber washers were employed to insulate the mercury cups in the stationary plate, and also the iron lift rod in the drop.

Since the 23 mercury cups and the 23 positive battery poles had to be connected with 8 lead-wires each, "eight wire forks" were interposed between the screw cups of the former and the bare ends of the latter. The "eight wire-fork" consisted of 8 short lead-wires from 2 to 10 feet in length; each of them carried a soldered-on screw cup at one extremity, and formed with the remaining wires a soldered metal point filed down to size of No. 10 wire, at the other end. Short pieces of lead-wire, provided with sufficient slack so as not to interfere with the free motion of the pin plate, connected the negative poles of the batteries with the respective pins. The iron lift-rod, fastened to and passing through the middle of the drop, ended in an iron cleat for belaying the suspension cord to, whose eye was slipped on, and by lashing secured over the middle of the horizontal cartridge. The latter was suspended from the middle of a light wooden frame, erected for this purpose on the roof of the battery-house. The cartridge itself consisted of a cylindrical tin shell, 2 inches in diameter and 6 inches in length, charged with $\frac{3}{4}$ pounds of dynamite No. 1 and two fuses, which were in electrical connection with the firing station, about 2 200 feet distant measured across the water, from which place the cartridge and frame could be seen. The auxiliary battery employed for firing the small dynamite cartridge consisted of 10 old bi-chromate cells, (4 good ones would have been sufficient). The total resistance of interpolar, *i.e.*, overground line and two of our standard fuses in derived circuit at moment of explosion, was 11.1 Ohms; current needed, 2×0.28 Webers. A Morse's key interposed near the auxiliary battery into the circuit was employed for touching off the small cartridge. At the appointed time this cartridge was exploded, and owing to the perfect

electrical arrangements, the simultaneous ignition of all the primers happened.*

Although the multiple circuit-closer is, with slight alterations, also applicable for electricity of the highest tension, yet the necessity of perfectly insulating the conducting wires, and the difficulty of accurate testing of the high tension fuses in the method of ignition by the electric spark, are alone sufficient objections against the attempt to use friction machines, etc., etc., for large and important blasts.

The choice between the electric sources reduces itself then, to an adoption of the voltaic battery or magneto-electric machines, whose methods of ignition being the same (current acting upon low-tension fuses) satisfactorily meet the above requirements.

III.—A COMBINATION OF BOTH METHODS; (Gen. Newton's method). At the great final blast at Hallet's Point, Hellgate, over 800 drill-hole charges were fired by the method of detonation by transmission, while about 4 000 mines were directly fired by the electrical fuses.†

THE MOST ADVANTAGEOUS GROUPING OF FUSES.

Capt. H. Schaw, Royal Engineers, states in a paper published in the "Journal of the Royal United Service Institution, 1865"—

"There are three general methods of connecting a number of charges with a voltaic battery to produce simultaneous explosion.

"Fig. 29, the continuous circuit. Fig. 30, the divided circuit. The first is the simplest, and, is suited to a large number of small cells, which

Fig. 29. Fig. 30. Fig. 31. is a troublesome battery; the second, the most certain, and is suited to a few large cells; but a large number of mines cannot be exploded thus."

"Fig. 31, shows a combination of the two systems which is the best for a large number

of mines; 21 were thus fired simultaneously in the demolition of the Keep at Corfu."

* A patent on the multiple circuit closer has been granted to its joint inventors, the writer and Mr. A. Doerflinger.

† Enumeration of recent publications treating on this subject:

Les Mines dans la guerre de Campagne par A. Picardat, Capitaine au 2d reg. du genie.

Theory of Simultaneous Ignitions, by Henry L. Abbot, Major of Engineers, Bvt. Brig. Gen'l U. S. A.

Preliminary Report of Lt. Col. John Newton, Bvt. Major Gen'l, U.S.A., of the Operations connected with the Destruction of the Reef at Hallet's Point (Hellgate) New York.

Let us inquire into the above conclusions. Put N = total number of fuses to be exploded simultaneously; n = number of fuses connected in continuous circuit in each group; $\frac{N}{n}$ = number of such groups united in divided circuit; E = electro-motive force of one cell in Volts; r = internal resistance of this cell in Ohms; x = number of such cells needed to cause the simultaneous ignition of all N fuses; R = resistance in Ohms of lead-wires connecting the battery (consisting of x cells coupled for "intensity") with the groups of fuses; f = resistance in Ohms of each fuse and its connecting wires at moment of explosion; and C = current in Webers, required to produce the simultaneous explosion of the fuses in one group—all such fuses being supposed equal in electrical resistance.

According to Ohm's law we have the equation: total strength of current needed to produce the simultaneous ignition of all the fuses arranged in combined circuit, (Fig. 31):—

$$\frac{N}{n} C = \frac{x E}{x r + R + \frac{n^2 f}{N}} \dots \dots \dots (1.)$$

Now, the maximum effect of a battery is attained, when the internal and external resistances are equal to each other, or when, in the case under consideration:

$$r x = R + \frac{n^2 f}{N} \dots \dots \dots (2.)$$

Substituting, alternately, this value of $r x$ and $R + \frac{n^2 f}{N}$ in Eq. 1, we get:

$$\frac{N}{n} C = \frac{E x}{2 x r} = \frac{E}{2 r} \text{ or } r \text{ the internal resistance of one cell:}$$

$$r = \frac{E n}{2 N C} \dots \dots \dots (3.)$$

$$\text{Also } \frac{N}{n} C = \frac{E x}{2 \left(\frac{R + n^2 f}{N} \right)};$$

hence the number of cells required;

$$x = \frac{2 N C \left(R + \frac{n^2 f}{N} \right)}{n E} \dots \dots \dots (4.)$$

Since the size of a battery depends upon its surface, or the total number of cells multiplied with the exposed surface of one cell, and since the latter is inversely proportional to the internal resistance of one cell, we can represent the total surface of the battery needed for the explosion

of all the fuses by total number of cells, divided by the internal resistance of one cell; whence from Eqs. 3 and 4

$$\frac{2NC \left(R + \frac{n^2 f}{A} \right) \frac{En}{En}}{2NC} = \frac{4N^2 C^2 \left(R + \frac{n^2 f}{N} \right)}{n^2 E^2} \dots\dots\dots (5)$$

The above formulæ, while it answers *case third*, the combined circuit, will meet *case first*, the continuous circuit, if $\frac{N}{n}$ becomes = 1; the expression then changes into

$$\frac{4C^2 (R + Nf)}{E^2} \dots\dots\dots (6)$$

And applied to *case second*, the derived circuit, in which case $\frac{N}{n}$ is changed into N , we have then, total number of cells divided by the internal resistance of one cell;

$$= \frac{4C^2 N^2 \left(R + \frac{f}{N} \right)}{E^2} \dots\dots\dots (7)$$

as representing the needed size of a battery which will simultaneously explode N fuses arranged in divided circuit.

Collecting these expressions (5, 6, 7), and dividing them by the factor $\frac{4C^2}{E^2}$, common to each, we derive for

$$\text{Case first} \dots\dots R + Nf \dots\dots\dots (8)$$

$$\text{" second} \dots\dots N^2 R + Nf \dots\dots\dots (9)$$

$$\text{" third} \dots\dots \frac{N^2}{n^2} R + Nf \dots\dots\dots (10)$$

representing respectively the comparative total surface of battery needed in each case.

Now, since $R < N^2 R$ and $\frac{N^2}{n^2} R < N^2 R$, it follows that the continuous circuit, *case first*, requires a battery having least surface; the combined circuit, *case third*, coming next, while the derived circuit, *case second*, demands the greatest total battery surface.

Again, an inspection of the expression for internal resistance of a cell belonging to the combined circuit, gives:

$$r = \frac{En}{2NC}; \text{ for case third,} \dots\dots\dots (\text{Eq. 3.})$$

$$\text{also, } r_1 = \frac{E}{2C} \quad \text{" first,} \dots\dots\dots (11)$$

$$\text{and } r_2 = \frac{E}{2NC} \quad \text{" second,} \dots\dots\dots (12)$$

Eqs. 11 and 12 (derived from Eq. 3 by replacing respectively $\frac{N}{n}$ by 1, and $\frac{N}{n}$ by N), show the necessity of using small cells in the *first case*, and larger cells in the *second case*.

The above deductions would be entirely correct were the same strength of current to suffice for the simultaneous ignition of the fuses of one group when arranged according to Figs. 29, 31, or Fig. 30. It appears, however, from numerous experiments made with different types of fuses, by no less an authority than Gen. H. L. Abbot,* that about three times the current which is needful to fire single fuses is required to simultaneously ignite a number of them when united in series, as, for instance, in Figs. 29 and 31.

Hence, according to the above, $\frac{C}{3}$ must be substituted for C in the equations deduced for the second case; performing this operation, we obtain,—the number of cells required, divided by the internal resistance of one cell

$$= \frac{4 C^2 N^2 \left(R + \frac{f}{N} \right)}{9 E^2} \dots\dots\dots (13)$$

Rejecting as before, for comparison's sake, the quantity $\frac{4 C^2}{E^2}$ in this expression, we have for *case second*:

$$\frac{N^2 R + N f}{9} \dots\dots\dots (14)$$

as indicating, comparatively, the battery surface requisite.

Let us now inquire, *when it is more economical to use the derived circuit, Fig. 30, than the continuous circuit, Fig. 29*; this will happen when $\frac{N^2 R + N f}{9} < R + N f$, and both will require the same battery surface when

$$R + N f = \frac{N^2 R + N f}{9} \dots\dots\dots (15)$$

Solving this quadratic equation for N , we get

$$N = \frac{4 f}{R} + \sqrt{\frac{16 f^2}{R^2} + 9} \dots\dots\dots (16)$$

which will answer in practice for a very small number, N , of fuses. For example, let $f = 2$, and $R = 1.4$ Ohms; then, $N = 5.71 + 6.45$, say 12.

We thus arrive (when employing cells of the same electro-motive force for the simultaneous ignition of N fuses), at the following results in regard to the three general methods of uniting a number of N fuses with the galvanic batteries:—

* U. S. Engineers, in charge of the Torpedo School at Willet's Point.

1°—If the total number, N , of charges is small (not exceeding 12, with fuses having a resistance of about 2 Ohms and a lead-wire resistance of nearly 1.5 Ohms), the derived and then the continuous circuit offer, so far as the needful total battery is concerned, the most economic means of connection.

2°—For a large number of fuses, the continuous circuit requires the least battery surface, but since, with an increase of N , the value of $\frac{N^2}{n^2} R$ (the square of the number of groups multiplied by the resistance of the lead-wire), in the expression $\frac{N^2}{n^2} R + Nf$; (Eq. 10), appears comparatively small with respect to Nf , and finally, when N is very large, $\frac{N^2}{n^2} R$ can without great error, be neglected; it follows that—

3°—When a very large number of charges is to be simultaneously fired, the continuous and the combined circuit demand the least total battery surface. But the combined circuit offers, in proportion to the number of its groups, greater security against misfire than the continuous circuit; for this reason, the use of the combined circuit for the simultaneous ignition of numerous mines is advisable, while the extra expense, for additional lead-wire, is small when compared with the advantage gained.

The next question to be considered is: *What is the best grouping of the N fuses when arranged in combined circuit?* From Eq. 3,

$$r = \frac{En}{2NC}; \quad \frac{N}{n} = \frac{E}{2rC} \dots\dots\dots (17)$$

The best grouping is obtained when the conditions expressed by this equation, are fulfilled.

We thus see, that the number of groups increases with an increase of the electro-motive force E , and diminishes with an increase of the internal resistance r , and the needful strength of current C . Also, that by adopting certain kind of cells (E and r), and fuses (requiring C), the best number of groups is given.

$x = \frac{2NC \left(R + \frac{n^2 f}{N} \right)}{En}$, (Eq. 4), shows that the number of cells is proportional to $R + \frac{n^2 f}{N}$ or the external resistance. This latter can be diminished by a reduction of R and $\frac{n^2 f}{N}$. R , the resistance of the lead-wire can be diminished by using as lead- and return wires, heavy copper wires of short length, (the application of the multiple circuit closer admits of this reduction in length of wire), or by using for each group

a separate set of lead-wires which start from the battery to their group of fuses arranged in continuous circuit. In the latter case, the external resistance is then expressed by the formula $\frac{R + nf}{N}$. If R compared with $\frac{n^2f}{N}$ in the expression $R + \frac{n^2f}{N}$, or, if R compared with nf in the expression $\frac{R + nf}{N}$ becomes relatively so small that it may be neglected,

then x becomes $= \frac{2nCf}{E}$; the number of cells required becomes proportional to the number of fuses in a group.

It might now be well, before closing this investigation, to consider the following question: *If the constants of fuses of types 1, 2, ... be given, viz.: their resistances f_1, f_2, \dots at the moment of explosion, and the currents C_1, C_2, \dots needful to produce their simultaneous ignition, when united in series, which type of fuses selected requires the smallest battery, and is hence the most economical?*

For any type 1, the most judicious grouping of the N fuses, to be fired by means of cells having an electro-motive force $= E$ and an internal resistance $= r$ is indicated by Eq. 17, which applied to this case reads $\frac{N}{n_1} = \frac{E}{2rC_1}$. From this equation, we find n_1 the number of fuses

of type 1 in one group, to be, $n_1 = \frac{2rNC_1}{E}$. And according to Eq. 4,

the number x_1 of cells required would be $x_1 = \frac{N}{n_1} C_1 \times 2 \frac{\left(R + \frac{n_1^2 f_1}{N}\right)}{E}$;

placing in this expression the above values of $\frac{N}{n_1}$ and n_1 we have:

$$x_1 = \frac{R}{r} + \frac{4rNC_1^2 f_1}{E^2} \dots \dots \dots (18)$$

Which means that the number of cells required, decreases with the value $C_1^2 f_1$ the product of the square of the needful current multiplied by the resistance of the fuse at the moment of explosion.

Gen. Abbot, gives (in his paper on the "Theory of simultaneous Ignitions;") the results of trials with types of fuses coupled in series. Type A consisted of $\frac{3}{16}$ inches of platinum wire, 0.0025 inches in diameter; type D of $\frac{1}{16}$ inches of platinum silver wire, 0.0015 inches in diameter; both charged with a priming of fulminating mercury. He found that type A required a current of at least 1.5 Webers, while with type D, one of 0.67 Webers was sufficient. Type A had a resistance of

0.82 Ohms, *type D* of 2.01 Ohms, at the instant of explosion. Combining these results with the deductions given above, we have

$$\text{Type A} \dots\dots\dots C_A^2 f_A = 1.5^2 \times 0.82 = 1.845.$$

$$\text{Type D} \dots\dots\dots C_D^2 f_A = 0.67^2 \times 2.01 = 0.902.$$

Which being interpreted means, that *type D* represents a fuse eminently superior to *type A* in reference to necessary size of battery, and that only about one-half of the number of cells needful for simultaneously firing N fuses of *type A* are required for N fuses of *type D* if $\frac{R}{r}$ becomes nearly zero.

Referring to Eq. 5, we see that the needful battery surface diminishes with an increase of E_1^2 indicating the importance of employing cells of a great electro-motive force.

FORMULAS REFERRING TO THE APPLICATION OF THE COMBINED CIRCUIT FOR SIMULTANEOUSLY FIRING N MINES. Each of the $\frac{N}{n}$ groups is provided with its own leading wires (of R resistance), connecting directly with the battery-house.

The general equation is:

$$\frac{NC}{n} = \frac{x \times E}{x \times r + R + \frac{N}{n}f} \dots\dots\dots \text{I. (Compare Eq. 1.)}$$

For the most economical numbers of groups:

$$\frac{N}{n} = \frac{E}{2rC} \dots\dots\dots \text{II. (Compare Eq. 17.)}$$

For the minimum needful number of cells coupled for intensity:

$$x = \frac{2C(R + \frac{N}{n}f)}{E} \dots\dots\dots \text{III. (Compare Eq. 4.)}$$

Remark.—For very exact calculations, Eq. III. is only applicable when, after entering the numerical values in Eq. II., both the number of groups $\frac{N}{n}$ and the number n of fuses in each group result in figures without fractional remainders.

Should this not take place, as will occur in nearly every instance, then the following equation, deduced from Eq. I., is the proper one to use in the calculation of the number of cells needed, naturally $\frac{N}{n}$ and n being inserted as whole numbers.

$$x = \frac{(R + \frac{N}{n}f)C}{E - \frac{N}{n}Cr} \dots\dots\dots \text{IV. (From Eq. I.)}$$

Again, sometimes, the following formulae will prove useful:

$$r = \frac{nE}{2CN} \dots\dots\dots \text{V. (From Eq. 3.)}$$

$$n = \frac{Ex - 2CR}{2fC} \dots\dots \text{VI. (From Eq. III.)}$$

$$N = \frac{E^2x - 2CER}{4rfC^2} \dots\dots \text{VII. (From Eqs. II. & VI.)}$$

A PRACTICAL EXAMPLE, ILLUSTRATING AN AVERAGE CASE IN THE PREPARATORY CALCULATIONS FOR THE FINAL BLAST AT HALLET'S POINT.

Given: $n=20$, deduced from practical considerations; $E=1.89$ Volts; $r=0.14$, $R=1.6$ and $f=2.18$ Ohms (mean values resulting from the writer's electrical measurements), $C=0.8$ Webers and increased for safety's sake.*

Found: $\frac{N}{n} = 8$ groups; $N=160$ fuses, and $x=37$ cells.

Applying Eq. II., and entering the above numerical values, we have:

$$\frac{N}{n} = \frac{1.89}{2 \times 0.14 \times 0.8} = 8.4, \text{ say } 8 \text{ groups; and } N=8 \times 20=160$$

fuses.

Using Eq. III., we find the required number of cells to be:

$$x_{\text{III}} = \frac{2 \times 0.8 (1.6 + 20 \times 2.18)}{1.89} = 38.27, \text{ say } 39 \text{ cells.}$$

And by employing Eq. IV., which is better suited for this case, we get:

$$x_{\text{IV}} = \frac{(1.6 + 20 \times 2.18) 0.8}{1.89 - 8 \times 0.8 \times 0.14} = 36.38, \text{ say } 37 \text{ cells.}$$

The correctness of the above calculation is easily proven by now entering these values in Eq. I., and since $37 > 36.38$ we should get a result a little larger than $8 \times 0.8 = 6.4$.

$$\frac{N}{n} C = \frac{37 \times 1.89}{37 \times 0.14 + \frac{1.6 + 20 \times 2.18}{8}} = 6.457$$

The above calculation would indicate that in this blast, one cell answers for about $\frac{160}{37}$, say 4 fuses.

* Gen. H. L. Abbot determined the value of the current required to ignite any number of our (silver-platinum, fulminate of mercury) fuses arranged in series at 0.675 Webers, and the resistance of the fuse at the moment of explosion at 2.01 Ohms.

DISCUSSION ON THE PRESERVATION OF TIMBER.*

MR. CHARLES DOUGLAS FOX.—In the paper by Mr. Clinton B. Sears,† a strong opinion is expressed against the efficacy of the creosoting process as a preservative against worm.

This matter must be of great importance in the United States, where such large quantities of timber are used for sea defences and piers, and I have therefore thought it desirable to present some little information upon the subject, and which, although not put into formal shape, may perhaps, in some way be brought under the notice of the Society.‡

I have used creosoted timber (Baltic redwood, impregnated with 10 pounds of oil of creosote per cubic foot) in works in a tidal river, where the worm is prevalent, and have up to this time (some seven years since the timber was fixed) found the creosote a complete preservative.

There is a difference of opinion among English engineers as to the extent to which this process is effective, but I think there can be no doubt that, where properly carried out, it is of great value, and greatly retards, if it does not entirely prevent, the action of the worm.

It is a very common practice to creosote railroad ties, both for home use and for export to India, where the process is found to be a great preservative against the white ant.

I may add, that to render the process effective, it is necessary—

- 1st. That the timber be previously well dried in open stacks.
- 2d. That the charge be carefully weighed in and out by an inspector.
- 3d. That the inspector should be present while the oil is being injected, to assure himself that the oil is of good quality, and that the suction-pipe passes well below the surface of the oil in the tanks.

* Referring to—CXXXI, Principles of Tidal Harbor Improvement. Clinton B. Sears. Vol. V, page 388. † Vol. V, pages 417, 418.

‡ Mr. Fox, in connection with his remarks, transmitted to the Society the following : Building Woods. The causes of their Decay and the Means of preventing it. G. R. Burnell. London. 1860.

Durability of Materials. Edwin Clark, with an Abstract of Discussion. Excerpt, Minutes of Proceedings, Institution of Civil Engineers. Vol. XXVII. London. 1869.

Memoire sur la Conservation des Bois a la Mer au Point de Vue Surtout de Leur Preservation Contre les Attagues du Taret. A. Forestier. Paris. 1868.

Nature and Properties of Timber, with descriptive Particulars of several Methods now in Use for its Preservation from Decay. Henry P. Burt, with abstract of Discussion. Excerpt Minutes of Proceedings, Institution of Civil Engineers. Vol. XII. London. 1854.

South Indian Railway Company, Specification for Sleepers. London. 1854.

Mr. S. B. Boulton, of a well-known firm of timber merchants in London, who has had, perhaps, as large an experience in this matter as any one, has, at my request, prepared some remarks on Mr. Sears' paper, in which he says :

"Mr. Sears, judging from his statements, has never seen any timber properly creosoted for the purpose of preserving it from the action or the 'teredo navalis,' and other marine boring insects.

"The process which he describes as the 'Robbins process,' and which has been attempted to be introduced here under various names, is thoroughly inefficacious, as it does not succeed in injecting a proper quantity of the creosote. Creosote injected by vapor has the effect of introducing a small quantity of the oil into the timber, which discolors it, indeed, but is not present in sufficient volume to protect from decay or attacks of insects, and 1½ pounds of oil to the cubic foot of timber would be altogether inadequate for either purpose.

"When Mr. Sears says—'it is possible that if a pile be thoroughly saturated with the oil to the extent of 10 or 12 pounds to the cubic foot, after an ample seasoning it may stand against the attacks of marine worms for a long time, but this, to my knowledge, has never been tried,' he appears to be unaware of the fact that this treatment has not only been tried, but has been carried out in practice to a great extent in England for thirty years, and very successfully, where properly applied.

"My firm has during that period sent out creosoted timber for harbor work in all parts of the world, and I have never heard of an instance of its failure. Cases of the failure of creosoted timber against the attacks of the marine insects have indeed been cited from time to time, but they have invariably arisen from imperfect penetration. I have seen several specimens of this sort during many years past, and even these specimens have in every case proved the efficacy of the creosoting process, as the worms have got into the uncreosoted part, but have never attacked the part which was impregnated with creosote.

"In reference to this subject, allow me to call attention to some remarks which I made at the Institute of Civil Engineers, May 12th, 1868, during the discussion upon Mr. Edwin Clark's paper on the 'Durability of Materials.'*

"The experiments in Holland, there referred to, were interesting, as they were tried with the greatest possible care by the Government, and with a bias which was at the commencement rather strong against the creosoting, or any other timber preserving process.

* See last note, preceding page.

"Neither the 'teredo navalis' nor the 'lunnonia terebrans' would attack timber which is efficiently creosoted.

"There is just a question whether Mr. Sears used creosote or heavy tar oil. There is an enormous series of hydro-carbons, but some of them containing petroleum have not been found efficacious for preserving timber. Petroleums, as you know, are never used in England for this purpose, but they were tried in Holland without success. It is possible that they may have been tried in America."

MR. CLINTON B. SEARS—Mr. Fox's letter is confined to an expression of confidence, as the result of his experience, in the efficacy of timber treated by creosoting, when the wood has been properly seasoned and carefully and sufficiently impregnated. He cites one case only. In this, 10 pounds of creosote to the cubic foot of timber had been used, and up to the present time—seven years—the wood has not been attacked by the teredo.

Mr. Boulton, for many years engaged in the manufacture of lumber and the treatment of timber by creosoting, expresses his entire satisfaction with the process, and states that after many years' experience, in which he has sent creosoted timber for use in harbors in all parts of the world, he has had no "failing cases," except in a few instances, and these due to abnormal conditions or circumstances.

The inference from some parts of his letter is, that in writing as above, I was culpably ignorant as to what had been done in this connection outside the field of my limited experience. This last I take in good part, as we view the matter from two different standpoints, and moreover when I wrote, I was *not* ignorant that creosoting had been carried on in England for some thirty years, that it was a very usual thing to treat timber to the extent of 10 pounds of creosote to the cubic foot, and that this was regarded by English engineers as effective against the teredo.

Mr. Boulton's own evidence must be taken as *ex parte*, as he is interested, but he supports his opinion by disinterested evidence in one of the pamphlets sent by Mr. Fox. The first three of these publications, I read several years ago, and also extracts from Forestier's Memoire, besides a number of other papers on the subject, both American and foreign.

Then why have I made so broad a statement in direct opposition to these writers? My remarks, as quoted at the start, were incidental, not pertaining particularly to the main subject, and were necessarily short and concise. It is but due to the above writers to say, that I should have

added the interpretation I now give, though at the time, I did not think it would be taken otherwise than as I intended.

I will say then, that my remarks embodied my opinions as to American practice and results only, were intended for American engineers, and were taken from a strictly professional point of view, viz., that every engineering project to be considered successful, must be not only efficient *de facto*, but must also be executed at a *reasonable cost*. This latter condition is more binding on American engineers than on those abroad, for there, both public and private corporations are much more liberal with their engineers than here, inasmuch as public sentiment demands that every work shall be thoroughly executed, and made as substantial as possible, even at the risk of bordering on extravagance.

That the contrary is the case in the United States is not the fault of our engineers, who greatly deplore the fact, and are striving to build up public sentiment to remedy the evil.

Strict economy, therefore, is essential, and my remarks were based on this condition, and I adhere to my opinion that the case of creosote *versus* teredo has not by any means been satisfactorily decided, for application in this country.

A multitude of pamphlets and reports (most of which I have read) have been published in the United States, endeavoring to convince engineers and the public generally that there are cheap and reliable methods of accomplishing the thorough work of Bethell, by using much less oil. The *cheapness* (?) consists in requiring an expenditure of from \$10 to \$12 per 1 000 feet, B. M., for from 1½ to 4 pounds only, of oil to the cubic foot, while the *reliability* has been time and again contradicted by the stern logic of experience.

One of the cheap methods used to replace Bethell's more thorough treatment, is the Robbins process, and I fully agree with Mr. Boulton that it is worthless as against the teredo. The average amount of creosote used is about 2 pounds to the cubic foot of timber, and as I have said, costs at least \$10 per 1 000 cubic feet, B.M. At this rate what would it cost to insure saturation to the extent of from 10 to 18 pounds of creosote per cubic foot? I cannot say, for as far as I know it has not been attempted to such an extent as would authorize a reliable estimate.

Now, turning to foreign countries, can it be said that even there the problem has been satisfactorily solved? I think not—to American engineers, at least. I have carefully read the above named English pamphlets, and some others, and have diligently translated all of Forestier's brochure ;

I find that there is considerable discrepancy as to the minimum amount of creosote thought necessary to insure immunity from the teredo. English engineers generally accept 10 pounds of creosote to the cubic foot; while Forestier, Engineer-in-Chief *des Ponts et Chaussées*, as the result of the official experiments carried on under his direction, says: "Nothing less than 18½ pounds per cubic foot, (300 *kilogrammes par metre cube de bois*), can be considered as reliable;" and in many of the experiments he used as high as from 22 to 34 pounds per cubic foot.

Taking all the experiments cited, two only, viz., twenty years at Sunderland, and thirteen years at Lowestoft, England, give tests of over ten years, while the average is seven years for the English, and only three years for the French experiments. Eight years have elapsed since the last of these particular pamphlets was published, and it would be interesting and instructive to hear again concerning the physical integrity of these structures.

It seems to be generally conceded that the life of timber submerged in sea water is, within certain limits, in direct ratio to the amount of creosote injected; that in pieces of average cross-section, the oil penetrates as an annular ring to a depth varying with the quantity; that it never completely saturates the timber unless the latter be very thin, or the amount of oil be very great, and this only in the soft woods; and that the invariable cause of failure to repel the attacks of the *teredo* has been a deficiency of creosote—of course, assuming in all cases that the quality of the latter, and the mode and execution of injection have been first class.

It appears, therefore, to be only a question of time as to when any given amount of creosote more or less, will be dissolved or washed out by the sea water, and the timber fall a prey to the enemy; if, however, the effective life of the timber can be extended *at a reasonable cost*, to, say twenty-five years, then we may consider the affair as sufficiently durable for ordinary purposes; but only ten or fifteen years of life increment, costing what it does, will hardly be considered satisfactory.

When we pass certain limits of time, we had better secure eighty or a hundred years' life at once, by founding our marine structures on cast iron piles or cribs, especially at the present rates for this material.

None of the above English publications say a word about the cost of treating timber to the extent of 10 or more pounds of creosote per cubic foot, nor in my researches elsewhere have I seen any estimates. We would like to hear from Mr. Boulton on this matter. Forestier gives

two estimates only. One of these shows that, in France, it cost \$10, coin, per 1 000 feet B.M. to inject 10 pounds of creosote per cubic foot ; while the minimum considered necessary there against the teredo, 18½ pounds per cubic foot, cost \$16, coin, per 1 000 feet. Query : what would this cost in the United States, taking into account the difference in cost of plant, labor and raw material ?

To be effective, the timber must always be treated *after* framing ; any cuts or gains made after the injection of the creosote will vitiate the results. This, to say the least, is inconvenient, and in many works would greatly increase the labor and consequent cost, and sometimes would be impracticable. This is a very serious drawback.

There are several other minor points I might dwell upon had I space : such as the relative values of creosoting for hard and soft woods ; inequality of penetration, due to varying densities in parts of the perimeter of a stick, as in half-round timber, &c., &c. ; which would leave doubts as to whether we can call creosoting a success.

This much at present, as to the processes in vogue abroad. They appear to be excellent and satisfactory, as far as they go, except as to cost. It was against the methods adopted in the United States that I made my quoted remarks, giving them only as my individual opinion, to be taken for what it is worth and nothing more. If I am wrong, I shall be most happy to be set right ; and even had my remarks been made in gross ignorance of what had been done in England in this matter, I should not particularly regret it since it has been the means of bringing this affair before the Society for discussion. I am not dogmatic, and will be only too glad to have my opinions successfully contested by a fair presentment of contradictory facts, as no engineer should care to establish a pet theory at the expense of ignoring reliable and desirable effects. I hope these remarks will be provocative of further discussion, and if my opinions be fairly assailed and overthrown I shall not complain, while others may profit by it.

In conclusion, I would say that Forestier's Memoire is the most exhaustive and satisfactory one I have yet encountered on this subject, and I commend it to those who wish to study up this matter.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

DISCUSSIONS ON SUBJECTS PRESENTED AT THE NINTH ANNUAL CONVENTION.

ON THE FAILURE OF THE ASHTABULA BRIDGE.*

GEN. GOUVERNEUR K. WARREN†:—I know of a circumstance that is interesting in considering the conflicting views of the engineer of the surviving locomotive and some of the passengers, as to whether the train was off the track or not, before the bridge broke. The case is as follows:

I am occupying a wooden frame building, built here about 160 years ago. In the spring of 1871, while sitting at work in a room 21 feet square, in the second story, something apparently of great weight fell upon the floor of the garret above me, shaking the house alarmingly and bringing down some of the ceiling.

This garret was stored with furniture, some of it consisting of earthen and iron ware packed in barrels. It did not belong to me, and I informed the owner of the building that if it was not removed at once, I should vacate the house. He removed the furniture, which operation I watched carefully, but could not discover that anything had fallen to produce the shock.

The matter remained a mystery to me until last spring, at which time the dangerous condition of the ceiling of the room compelled me to remove the plaster. I then found the main beam which supported the garret floor, broken short off, the ceiling being supported mainly by the garret floor.

The old style of floor beams was very different and very inferior to the present one. One large beam of oak, a foot square in section, went through

* Referring to—CXXXVII, The Failure of the Ashtabula Bridge. C. Macdonald. Page 74.

† Presented January 18th, 1877.

the middle of the floor or ceiling, and small joists rested on this and the sides of the room. This main beam always sagged down so as to crack and otherwise deform the ceiling beneath, and some inconsiderate modern carpenter invented a way of straightening this beam by sawing it through about two-thirds from the top; then forcing it up from below by strut and wedges under the middle where the cut was, he made the beam come up to the old level at the middle part. The cut was thus opened like a wedge, into which an iron wedge was driven; the strut below was then removed. The timber being very old, this flexure at the cut ruptured many of the fibres of the wood, which thereafter had to sustain a great tension—like the bottom chord of a bridge truss, whose height was small in comparison to the span, 1 in 21, the introduced iron wedge acting as the top chord. It has been a wonder to me, how the beam stood what it did, which was probably due to the ends of the beam bracing against the sides; and it broke as short as would a piece of steel wire under a breaking strain of tension. It, as it were, snapped, and the giving way was so sudden, that the shock was precisely that of something falling.

If then, the bottom chord or a tie rod snapped, or if one after the other did, at Ashtabula, the sensation in the cars would be that of a car jumping off the track on to a beam. The engineer, a little removed from the place where the break occurred, might not feel such shock, which he could hardly have failed to do, if the shocks had been the striking of the car wheels against floor beams, which would have made a jerk upon the engine.

Fig. 31.

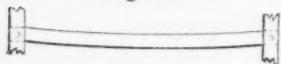


Fig. 32.



The following shows the rectified ancient floor beam, as practised here in Newport; Fig. 31 being as before, and Fig. 32 as after, straightening. After this is done, they hew off the smaller convexity on each side, or level

up with furring pieces, and plaster it over again,—as well hidden a human trap as ever was set for wild beast.

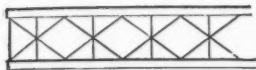
MR. EDWARD S. PHILBRICK*:—I had an assistant† at Ashtabula, sent to inspect the wreck as it was removed, from whom I have some facts which may be of interest. He states that:

1^o.—The bottom lateral bracing was composed of flat bars, $2\frac{1}{2} \times \frac{1}{2}$ inches, applied in panels of 22 feet, and attached to chords by hooks in

* Presented February 6th, 1877. † Mr. A. H. Howland, who has had considerable experience with various styles of iron bridge plans.

ends of ties which fitted recesses in angle blocks. The struts were railroad track bars, and were applied at points intermediate between those points where the lateral brace ties were attached. The latter had no

Fig. 33.



adjustment for length. Thus, if these ties had been screwed up, the only effect would be to put the chords out of line, and make them assume a zigzag form; and all lateral forces arising from wind or trains passing, which would operate to break up the alignment of the chords, would not be controlled by this lateral system—its only effect seeming to be to pull the structure into a serpentine line.

2°.—The only attachment for the transverse vertical bracing was by hooks fitting into the angle blocks, and held in place by a $\frac{3}{8}$ -inch tap bolt. These ties were adjustable by turn-buckles, and were applied at intervals of 22 feet at same points as the lateral bracing, and therefore intermediate between the bottom transverse struts.

3°.—The top chords were composed each, of five lines of 6-inch I beams, side by side. They were clamped or held together only by two $\frac{3}{8}$ -inch bolts, per panel.

4°.—The lugs which were cast on the angle blocks, for the purpose of holding the main braces in place, were mostly chipped off when the braces were turned on their own axes, before using the bridge, and did not fit the new position of braces. Some of these, showed several inches change of position when last painted. These braces of the main trusses were clamped at their intersection, *i. e.*, where the main brace intersects the counter, by two 1-inch bolts only, not passing through, but outside, as a yoke.

As to position of wreck, the first set of braces in the south truss at west end, were found nearly parallel with the face of abutment, with their upper ends *under* the north bottom chord, which bent them somewhat where falling on them. The bottom chords were nearly parallel, the south one lying near centre line of bridge, both having moved about 8 feet north, and both had moved in falling, about 8 feet east, *i. e.*, end-wise, folding up their east ends against the east abutment. The first set of vertical rods in the south truss, beginning at its west end, had their top angle block slid quite down to the bottom chord, and lay spread out fan-shaped towards the abutment. The next set had the top angle block also down, but were bent to the south. The third set of rods bent over to the north, with top angle block nearly down. Others, with angle blocks at top

end, bent north till near east end of bridge, where they bent south again. All the rods of the north truss were bent to the north. On the top angle block of the south truss, second from its west end, the paint showed that the chord bars which abutted against this block had a bearing on one edge only, as if the bar in the first panel were sprung convex to the south, while others in same line in next panel eastward, indicated being convex to the north. All the I beams, forming the top chords, main braces and floor beams, were dragged out as soon as possible after the disaster, and irrevocably mixed. But the position of the bottom chords, vertical rods, angle blocks, &c., as given above, taken together with the evidence of the engine-men, seem to indicate these points, viz.:

The failure began by the buckling or displacement of either main braces or top chords at about the second and third panel from west end of south truss. The angle blocks belonging to the top chord at this point were all slid down the whole length of the rods, which passed through them, 8 in number at each block, and this could only happen *after* the braces ceased to support the angle blocks, and *before* the rods had left their vertical position to any extent. The second set of rods were bent over to the south, possibly getting started in that direction before the train began to fall. The floor must have sunk first on the south side, for that side only was loaded, and the train was piled up on the south side. The lateral reaction of its fall, from the sloping deck, would push the rest of the bridge north, as it did go, even against the powerful wind. The west end of the bottom chords falling first, as the north truss leaned over, the whole hinged for an instant while falling, upon the east end, pulling the bottom chords eastward, as found.

Diligent search was made for evidence of derailment, but none found—a few wheels were broken and cracked, but nothing to show whether they were sound or unsound before the fall. They showed marks of a good deal of heat, and could easily have received all the injury they show, even more, by the fall of 70 feet and subsequent roasting. Some 40 ties were found, but no wheel marks on them.

There seems to have been sufficient elements of instability in the disjointed and poorly connected top chords and main braces to account for the disaster, even without any derailment of wheels. This floor is described to be better than that upon many other bridges which I have seen carry trains over with wheels off the rails without injury, though an increased thickness of planking would have contributed much to this one, in such cases.

MR. ALFRED P. BOLLER.*—As an engineering work, the Ashtabula bridge has lost much of its interest, so thoroughly have the particulars and details concerning it, been published. We all know it to have been a conglomeration of errors, and principally astounding in its longevity. Why it lasted a week after the staging was knocked out, can only be answered by reference to the doctrine of "special providences." That it lasted a dozen years, is a superb tribute to the value of iron in bridge construction, showing the torture that material will stand before the penalty is paid, that nature exacts of ignorance. Without moralizing over the design, ignorantly conceived and faultily carried out, and one that any bridge expert would have condemned after less than five minutes inspection, the lesson of the disaster is of the highest importance to the whole community.

In the first place, the most pointed lesson to be learned, is to recognize the limitation of human powers, and the necessity of men keeping within the bounds of their exact knowledge. Too many of us, in the belief that we are "engineers," feel it incumbent in our professional pride, to lay claim to all knowledge covered by the term *engineer*, and are far too ready to assume responsibility in any engineering matter brought to our attention. The public aids in keeping up the flattering fiction that an engineer must, of course, know all of engineering, and the same individual is often called upon for sage advice on subjects of the most diverse character. The modern engineers' experiences are far from being so varied as those of a generation before him, and it is now almost impossible to be an expert in more than two or three subdivisions of the profession. As knowledge increases, it necessarily requires division of labor, which must become more and more subdivided as each class accumulates new facts and experiences.

This is a law of civilized development, and being such, it surely need be no weakness for any of us, to admit that there are many things we do not know, or that others know much better. A man may have general ideas concerning a locomotive, pump or a bridge, but how absurd it is for him to undertake to design one or the other, when there are those who have given their lives to the study and practice of each speciality, and are ready at a moment's call to furnish any desired information. A knowledge of general principles, is far from being a knowledge of design, in a practical sense. The former can be learned from books, the latter the books teach only in small part.

* Presented April 4th, 1877.

The time has about arrived for us to overhaul our classification, whereby the term engineer would have some prefixes, recognizing the divisions of labor under which the profession is constantly developing. The result of such classification, would be that engineers would be compelled to range themselves under it, sooner or later, according to their knowledge, and the public would very quickly learn to pick out a professional advisor for his special knowledge in the department of engineering wherein information is sought.

Then, if men wanted a locomotive built, they would go to a locomotive engineer, or to a hydraulic engineer for hydraulic works, or to a bridge engineer for their bridges, or to a locating engineer if they wanted a railroad laid out, and so on. If this principle had been recognized a dozen years ago, Mr. Stone would probably have not undertaken to design the Ashtabula bridge in iron, but would have called in the best attainable expert of the day. Mr. Whipple would have built him a good bridge, so would Mr. Linville, or Mr. Fink, men who at that day had become experts, and peculiarly so the former gentleman, who thirty years ago, was not far behind our theories of to-day. And so I read the lesson of the Ashtabula disaster: not to be actuated by a false pride to undertake to do things we do not understand, but rather exercise worldly common sense in calling to our aid, those whose studies and experience have made them experts in the thing to be done.

The second and only other point of interest in this discussion, covers the prevention of such disasters for the future. The first impulse with most people, when any thing goes wrong is to devise some legislation that will prevent its repetition. There is a natural feeling that government is paternal in its character, without whose interference the affairs of state and of corporations cannot prosper. Doubtless, laws are necessary, but they are evils at best, out of which frequently grow greater difficulties, than those they were devised to overcome. Laws beget laws, and their tendency is toward interference with wholesome, legitimate development. I am a believer in legislation only as a matter of last resort, preferring that evils should work their own correction if possible, from within outward, rather than be suppressed from the outside. The former is natural and proceeds from self interest if not from higher motives, while the latter is interfering, and therefore creative of antagonism. These remarks I conceive, apply with great force to railroad interests, which in some parts of the country have been badgered nigh unto death with well-meant legislation. The

object of the owners of a railroad is to make it pay, and they are well aware that to do so their road must be kept in a good, that is, safe condition. They know full well, for instance, that a bridge disaster means a drain on their treasury, and in the case of the Lake Shore railroad, this drain, growing out of but one disaster, will probably not be covered by less than a million dollars—all things considered, damage to property, to life and injury to business.

Bad bridges are built through ignorance, but their number is wonderfully decreasing year by year, and sad as the accident at Ashtabula was, the notoriety of it, with all the attending circumstances, has reached every railway official in the land, and they are beginning to realize that iron bridges are dangerous unless intelligently built, that such are constructions which require care and that the foreman of bridge repairs is not the proper person to pronounce upon their safety or manner of performing their work. I venture to say that there is hardly a railway in the country that has not been inspected in *some* way, as to its bridges, since last December. It is to be hoped that experts have been employed, without whose knowledge, inspection will not amount to much. There are, doubtless, still standing in daily service, iron bridges that are dangerous for the safe passage of trains, from insufficient transverse bracing, unguarded floors, or weak ones, if in no other particulars, and it will be a very great economy for those roads that have not done so, to employ at the earliest moment, some one who *knows* the points to be looked after.

This, then, in my judgment, is all that is needed for the prevention of bridge accidents: "Have all bridges examined at least once yearly, by some expert who has had experience as a bridge builder, as well as being a theoretician." Railroad managers can rely upon it, if such a policy is honestly carried out, that in less than two years there need not be a dangerous iron bridge in America, or a bad one built hereafter.

As an illustration of the idea that, where possible, it is always best to allow evils to work their own cure through natural channels, take the recent action of the Reading Railroad Co., with reference to the Brotherhood of Locomotive Engineers, an organization that prosperity and discipline had made unbearable. Massachusetts and some other States felt driven to legislation to protect the roads and the public from the exactions and annoyances of the arrogant engine drivers. With one general order, the vitals of the Brotherhood have been pierced by the Reading Co., and now that Mr. Gowan has shown how to "stand the egg on its

end," doubtless the other roads will follow suit, and it cannot be long before the Brotherhood will be among the things that were. Such action, on the part of this railroad, has been worth volumes of laws, in that the moral effect of establishing one's own mastery, enforces a respect that laws are powerless to accomplish, not only in relation to the Brotherhood, but in all its relations to labor.

Holding then, to the doctrine of the least possible governmental interference, and also to the principle that men should be selected for positions of responsibility on the score of their experience and training in any special direction, it is impossible for me to approve of the "Adams" bill, introduced into the House of Representatives by Mr. Garfield,* covering the appointment of army engineers as a commission to investigate railway accidents, and report on the most approved means of preventing the same. On its face it appears innocent enough, not likely to interfere with the liberties of corporations, but it authorizes the Commission to report upon the means of prevention of accidents, which would be a useless provision, unless it was intended to frame laws based thereon as to what companies should or should not do, growing out of this bill. I can imagine various army inspectors appointed for the different departments of railway service, which would very much interfere with the railway interests. It is a very serious objection to the bill, that it is committed to the hands of army engineers to carry out its provisions. These gentlemen are not fitted by occupation or training to sit in judgment on railway affairs, complicated at best, and involving for their proper estimation, a knowledge gained from experience. As well appoint civil engineers as experts to sit upon the best method of prevent-

* BILL TO PROVIDE FOR A MORE THOROUGH INVESTIGATION OF ACCIDENTS UPON RAILROADS (H. R. No. 4538), INTRODUCED BY JAMES A. GARFIELD, FEBRUARY, 1877.

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled:

§ 1. That the President of the United States is hereby authorized and requested to appoint a Board of three Commissioners, who shall be Officers of Engineers of the Army, to inquire into the number, causes and means of prevention, of accidents on railroads in the United States, the number of persons killed or injured thereby, and the most approved means of preventing the occurrence of the same; and it shall be the duty of said Commissioners to hereafter investigate such accidents on railroads as may in their judgment be accompanied by circumstances of an unusual or unexplained character, and specially report upon the same.

§ 2. That the Commissioners appointed under this Act shall, in addition to their pay as officers of Engineers of the Army, receive compensation for actual travel and other necessary expenses incurred in the duties herein designated.

§ 3. That, in addition to all special reports from time to time made, the Commissioners herein provided for, shall, at the close of each year, forward to the Secretary of the Treasury a general report upon the subject of accidents upon railroads in the United States during that year, which report, together with any special reports which the Commissioners may have made during such year, shall be submitted to Congress.

ing the bursting of artillery, or the construction of a fort. So far as the bill covers the compiling and classification of railway accidents, or the sitting of the Commission as a coroner's jury, the army engineers could do that work well and the results would be useful as statistics, but the recommending of preventions, involves the experience and judgment of the practical railroad manager and constructor, to whom alone that department of the bill should be allotted. I think the bill could be modified so as to operate usefully, if confined to the simple idea of compiling statistics, a class of work the government ought to do a great deal more than it does, but I most emphatically say "hands off" at any indication of government non-experts interfering with our railroads or other institutions.

MR. SQUIRE WHIPPLE.*—Were I a juror upon an inquest in the case of the Ashtabula bridge, my verdict would be that the structure owed its destruction to an excess of 6-inch I beams used in the construction of its compression members. If the old man in the fable, who broke the bundle of sticks one by one, to enforce upon his sons, the lesson of strength in union and weakness in division, had used a solid timber as against the bundle of small rods, the illustration would have been still more striking; and, if half the material in the braces, and a little more than half that in the upper chord of the bridge, had been used in the form of Phoenix columns, or other well proportioned hollow trunk form, the result would, undoubtedly, have been vastly better.

It seems to be quite evident, as well as generally conceded, that the failure commenced at the second panel length from the west abutment, by the yielding to deflection of the second set of braces of the south truss; possibly preceded and partially induced by the breaking of one of the lugs rising upon the cast iron angle block. It may never be known which of these two results preceded the other, and practically it is of little consequence that it should be, as that mode of construction is not likely to be repeated, and if there are other specimens of the same kind of practice, the catastrophe in this case will call attention to them, and a like result, in regard to them, will be averted.

The power of the I beam to resist flexure at right angles with its web, is, of course, very small; so large an excess of its material being in and near the web, and offering slight resistance to flexure. Allowing half the material, 1 inch, and the other half, 4 inches diameter (perpendicularly to the web), giving an average of $2\frac{1}{2}$ inches, the length of the brace,

* Presented April 4th, 1877.

about 22 feet, would be 105 diameters, and the half-length $52\frac{1}{2}$ diameters. Now, the clamping of the braces in the centre could only prevent flexure in opposite directions by the several pieces. In case half the pieces were inclined to deflect northward, and the other half southward, in equal degrees, the tendencies would be neutralized, and the lengths practically reduced to $52\frac{1}{2}$ diameters. But, were all, or a majority of the pieces inclined one way, as they almost certainly would be, the opposite tendency of the minority (if any) would be overcome, and these being forced past the point of equilibrium, they would all tend the same way, and the brace to all practical intents would have a length of 105 diameters, with an absolute power of resistance considerably less than that required for the actual stress shown for the parts in question, by Mr. Macdonald's strain sheet.

The section of iron in the six pieces constituting this brace, was about 58 square inches, and we see that under conditions easily supposable, and scarcely improbable, failure could have been predicted with certainty; whereas one-half of that section in the Phoenix column of 10 or 12 inches in diameter, would have sustained the compression with as near an absolute certainty as is attained in practical matters of this kind.

It seems hardly necessary to criticise this structure in detail, or to great length. As Mr. Macdonald pertinently remarks,* the plan is exceptional in design and execution, and so radically faulty in many particulars as to be at once discarded by every intelligent bridge engineer of the present day. And that it endured so long with faults so obvious, should serve to confirm rather than impair public confidence in the safety of well designed and constructed iron bridges, such as are built by most of the prominent builders of this country.

I would remark further, that while the general arrangement and outline of the trusses of this bridge (the trapezoid with verticals and diagonals in the web) was one of the best general forms in use, and while the material was in even super-abundance, and of good quality, a weak and inefficient structure was produced by neglect of certain important principles which should never be lost sight of, in arranging the details of a bridge truss. It was faulty to introduce oblique thrust members and tension verticals instead of the reverse, in the web. By this means the thrust members were unnecessarily increased in length, and the efficiency of the material diminished. This remark is not applicable in

* Page 85.

case of trusses *without* vertical members, in which the oblique thrust members are prevented from deflection in the plane of the truss by connection with tension members at the crossings, and from deflection transversely to that plane, by the tension of crossing members as well as by greater diameter transversely; whereby the efficiency of material in thrust members is preserved.

But it was a much greater fault, and probably the one mainly leading to the fatal result, to divide the material of the braces and upper chord into 5 or 6 slender bars, affording but little mutual support laterally, instead of consolidating a smaller amount of material in single efficient members of large diameter and lateral stiffness.

The practice, moreover, of distributing the floor beams over the upper chord between supported points, imparting a very considerable lateral strain in addition to the compression which it is the main function of that chord to sustain, is a practice meriting disapprobation and discouragement.

MR. CHARLES HILTON.*—As has been shown by Mr. Macdonald, it is quite evident that the failure of the bridge began in the second panel division from the west end of the south truss, by the bending of the upper chord or of the second main brace or perhaps of both simultaneously, although the iron in both chords and braces at this place was much less severely strained than corresponding members in other parts of the bridge; and hence it would appear, that there must have been some defect in the material or workmanship at that point, or some other assisting cause aside from the ordinarily recognized strains due to the load upon the bridge.

Mr. Macdonald thinks that the explanation may be found in the weakness of the lugs on the second top-chord angle block. As described, this angle block had two lugs cast upon its upper surface, through which the horizontal component of the strain on the main braces was transmitted to two beams of the upper chord. On gathering up the wreck, one of these lugs was found to have been broken off, as was the case with the lugs on many of the other angle blocks, the fracture in this and several of the others showing defects in the casting, as has been explained. But as the other lug upon this angle block remained intact, I find myself unable to understand clearly how the breaking off of one of

* Presented February 21st, 1877. The following, were all read at the Ninth Annual Convention.

the lugs could have caused the bridge to fall while the other lug remained unbroken. The first effect of breaking off one lug, would be, to throw a double strain on the other, and it seems to me that it must have been the next thing to give way, in the course of destruction.

In seeking for a more probable auxiliary cause of failure, I have been led to consider some circumstances that appears to have either escaped the notice of other investigators, or have not considered by them entitled to serious consideration.

Ashtabula station is about 1 200 feet west of the west end of the bridge, and about 500 feet west of the station is a level railway crossing. The Ohio State laws require all trains to come to a full stop not less than 400, nor more than 800 feet from all level railway crossings, and hence all trains going west on the Lake Shore & Michigan Southern R. R. must be brought to a stop, with the engine from 900 to 1 300 feet west of the west end of the bridge, and it seems altogether probable that in most cases the brakes have been applied while the trains were upon the bridge. Let it be borne in mind, that all such trains were upon the south track, one rail of which lay over the middle of the south truss. Now whenever the brakes were applied to the train, a force equal to the friction upon the wheels was transmitted to the track, tending to force the rails to move in the direction in which the train was moving.

Assuming the connections of the bridge floor and track with the roadway and track westward from the bridge to have been perfect and unyielding, no effect would have been produced upon the framework of the bridge; but on the other hand, if the wooden platform and rails on the bridge were free to move at the abutments in the direction of the rail line even to a small extent, a force equivalent to the friction of the brakes on the wheels upon the bridge must be transmitted to the abutments through the framework of the bridge itself. I consider it not at all improbable, that generally the connection between the bridge floor and the track with the roadway and track at either end was so far imperfect and yielding that a considerable portion of the force exerted through the brakes upon the track, had to be resisted by the bridge truss and mainly by the top chord and main braces, of the west half of the bridge—the maximum stress from this cause occurs near the west end of the truss. From what I have been able to learn in relation thereto, I am inclined to the belief that the engineer of the second engine of the ill-fated train, applied the air brakes just as he reached the west end of the bridge, and that the increased stress thereby produced upon the upper chords and main

braces near the west end of the truss may have been sufficient to have been the immediate cause of failure at that moment, and at that particular place.

If the views I have expressed have any foundation in fact, the question of the effect of frequently-repeated breaking up of trains on under-grade bridges certainly deserves more attention from engineers than it appears to have received hitherto.

MR. THOMAS C. CLARKE.*—Most of the arguments in favor of bridge inspection proceed upon the supposition that railway bridges alone need it; some think that the railway officers themselves can be trusted to do this, while others think inspection should be placed in the hands of independent persons.

The truth is, that railway bridges are, as a whole, much more carefully looked after, and are far safer than highway bridges. Inspection, to be valuable, ought to be general, and cover all kinds of bridges. Mr. Macdonald's *reductio ad absurdum* argument,† would abolish boiler inspection and every other kind of inspection now in use, which, feeble as it is, is better than nothing.

But the most important requisite, in my opinion, is that inspection of bridges should be made according to an uniform system. As it is now, the State Legislatures, under the spur of the panic derived from the Ashtabula disaster, are passing laws providing for inspection under different systems, and with different penalties.‡ The enforcement of these laws will be placed in the hands of men of very varying qualities.

* Presented March 20th, 1877. † Page 84.

‡ The following is one of the bills proposed:

A BILL TO SECURE GREATER SAFETY FOR PUBLIC TRAVEL OVER BRIDGES.

§ 1. *Be it enacted by the General Assembly of the State of Ohio.* That all railroad bridges hereafter erected and designed or used for public travel, except those provided for in § 17 of this Act, shall be built to carry, for usual loads, not less than the following, in addition to their own weight, namely: bridges having a span of 7½ feet, and under, 9 000 pounds per lineal foot for each track; those having a span of from 7½ to 10 feet, 7 500 pounds per lineal foot for each track; those having a span from 10 to 12½ feet, 6 700 pounds per lineal foot for each track; those having a span from 12½ to 15 feet, 6 000 pounds per lineal foot for each track; those having a span from 15 to 20 feet, 5 000 pounds per lineal foot for each track; those having a span from 20 to 30 feet, 4 300 pounds per lineal foot for each track; those having a span from 30 to 40 feet, 3 700 pounds per lineal foot for each track; those having a span from 40 to 50 feet, 3 300 pounds per lineal foot for each track; those having a span from 50 to 75 feet, 3 200 pounds per lineal foot for each track; those having a span from 75 to 100 feet, 3 100 pounds per lineal foot for each track; those having a span from 100 to 150 feet, 3 000 pounds per lineal foot for each track; those having a span from 150 to 200 feet, 2 900 pounds per lineal foot for each track; those having a span from 200 to 300 feet, 2 800 pounds per lineal foot for each track; those having a span from 300 to 400 feet, 2 700 pounds per lineal foot for each track; those having a span from 400 to 500 feet, 2 500 pounds per lineal foot for each track; and in all bridge trusses of whatever length, the several members in each panel shall be so proportioned as to sustain, in addition to its share of the uniform load as above stated,

I should much prefer to rely on Mr. Garfield's bill, if it contained an additional provision, that his Board should, first of all, meet and take evidence from engineers, particularly those expert in iron bridge construction and establish upon the basis of that evidence, certain general rules to apply to all bridges, both railway and highway; such as, *first*,—the least load to be provided for, on different lengths of spans, and on parts of the same span; *second*,—the margin of safety; *third*,—the mode of connecting the parts together, so as to form a whole as strong as those parts, and *fourth*,—such mode of construction as would best prevent deterioration and decay.

All of this, I believe, could be expressed in such words, as would, while not fettering the skill of designers, yet would increase the safety of the public. These rules should be published and communicated to the different authorities, state, municipal and railway. As to what action should be taken after that, in regard to penalties, etc., not being a lawyer, I shall say nothing.

As an engineer, however, I wish to record my opinion, that inspection should be in the hands of independent parties, should cover all kinds of bridges, and should be under one uniform system, based upon evidence taken from experts in bridge construction.

such concentrated panel load as is herein provided for a bridge of a length equal to the length of the panel.

§ 2. Every railroad bridge shall be so constructed as to be capable of carrying on each track, in addition to its own weight, two locomotives coupled together, each weighing 91 200 pounds, on drivers, in a space of 12½ feet for each locomotive, and said locomotives to be followed by cars weighing 2 250 pounds per lineal foot, covering the remainder of the span; and all railroad bridges shall be so projected that the loads above mentioned in § 1 shall not strain any part of the material in such structure, beyond one-fifth its ultimate strength.

§ 3. All bridges hereafter erected and designed or used for public travel, on any highway or wagon road, shall be constructed to carry, beside their own weight, not less than the following standard loads, namely: city and suburban bridges, and those over large rivers, where great concentration of weight is possible, and on highways in manufacturing districts, spans of 30 feet and under, 110 pounds per square foot; spans from 30 to 50 feet, 100 pounds per square foot; spans from 50 to 75 feet, 90 pounds per square foot; spans from 75 to 100 feet, 80 pounds per square foot; spans from 100 to 200 feet, 75 pounds per square foot; spans from 200 to 400 feet, 65 pounds per square foot: on all other highway or road bridges, the standard load shall be not less than 100 pounds per square foot, in spans of 30 feet and under; 90 pounds per square foot, in spans from 30 to 50 feet; 80 pounds per square foot, in spans from 50 to 75 feet; 75 pounds per square foot, in spans from 75 to 100 feet; 60 pounds per square foot, in spans from 100 to 200 feet; 50 pounds per square foot, in spans from 200 to 400 feet. The floor-beam strength for each floor-beam for each wagon-way of city bridges, and those near large manufactories, shall not be less than 13 500 pounds; for other bridges, not less than 11 250 pounds.

§ 4. In the construction of all bridges for public travel, either for railroads or common wagon ways, the stress on any material used in the construction of the bridge, in carrying the maximum load for which such bridge is designed, shall not exceed the following, namely: for the best quality of wrought iron, in tension long bars or rods, 10 000 pounds per square inch: for short lengths, 8 000 pounds per square inch, and against shearing force, 7 500 pounds

MR. ROBERT BRIGGS.—In common with most engineers, I have taken a lively interest in the inquiry into causes of failure of the Ashtabula bridge. I wish to call attention to some considerations which ought to be stated, and possibly should have a mathematical investigation.

The inquiry of strength, so far as Mr. Macdonald's paper is concerned, is confined to the truss strength to carry vertical loads, but as suggested by Mr. Philbrick, the deflection of the floor beams would render the load far from vertical; and it seems to me that a full discussion of the strength and deflection of these floor beams, and the effect of the strain, from this deflection or effort upon the upper chords, could be computed with much accuracy. No mention is made of the condition of the floor beams after the accident, and the presumption is that they were found to be unharmed and straight, but it is difficult to conceive of a floor of 6-inch deep beams, spaced 3 feet 8 inches from centre to centre, as suitable to carry the rolling load of a train and 2 engines, besides the floor load. Mr. Macdonald gave the floor beams, 6 inches deep with 4-inch flanges and 9.6 inches section, which gives 0.785 as average thickness of the beams, web or flange. A rough computation, where I do not want accurate data of exact place of application of load and of weight of floor, would give the loads on these floor beams very close upon the limit of permanent elasticity, and would throw more than double

per square inch; and for the best quality of wrought iron, in beams either square or cylindrical in section, in compression, the following, namely: beams having a length of 10 diameters, 10 000 pounds per square inch, with square ends, and 7 000 pounds with round ends; beams having a length of from 10 to 15 diameters, 9 000 pounds per square inch, for square ends, and 6 500 pounds for round ends; beams from 15 to 20 diameters, 8 000 pounds per square inch, for square ends, and 6 000 pounds for round ends; beams from 20 to 25 diameters, 7 500 pounds per square inch, for square ends, and 5 500 pounds for round ends; beams from 25 to 30 diameters, 6 800 pounds per square inch, for square ends, and 5 000 pounds for round ends; beams from 30 to 35 diameters, 6 000 pounds per square inch, for square ends, and 4 000 pounds for round ends; beams from 35 to 40 diameters, 5 000 pounds per square inch, for square ends, and 3 500 pounds for round ends; beams from 40 to 50 diameters, 3 800 pounds per square inch, for square ends, and 2 500 pounds for round ends; and beams having a length from 50 to 60 diameters, 3 000 pounds per square inch, for square ends, and 2 000 pounds for round ends. If iron inferior to the best quality be used, either in tension or compression, the stress on the same shall be proportionately less than the foregoing standard for wrought iron of the best quality.

§5. Cast iron may be used in the construction of bridges, in compression only, and in lengths not exceeding 20 diameters, at the same stresses as those prescribed for wrought iron by this Act; and in shapes other than square or cylindrical, whether wrought or cast iron be used, the stresses shall vary accordingly.

§6. Where wood is used in the construction of any such bridge as aforesaid, the greatest allowable strain shall not exceed the following, namely: for oak in tension, 1 200 pounds per square inch; for pine, 1 000 pounds per square inch; and in compression, for oak beams of 10 diameters, 1 000 pounds per square inch; and for pine, 900 pounds per square inch; for oak beams, from 10 to 20 diameters, 800 pounds per square inch, and 700 pounds for pine; for oak beams, from 20 to 30 diameters, 600 pounds per square inch, and 500 pounds for pine; and in oak beams of from 30 to 40 diameters, 400 pounds per square inch, and 300 pounds for pine.

the compression strain on the inner struts and chords of the truss. If it could be shown that the beams, at or near the place of apparent failure of the truss were sprung, the probability of the failure having proceeded from the *rocking* of the wide upper chord lines, might be asserted; without such evidence, it may be assumed, the effort of deflection must have produced, as Mr. Philbrick says, "a transverse strain on the two inside members of the top chord," and besides this, a severe excess of load and diagonal strain on the inside members of the struts.

The application of Gordon's formula to these sections is scarcely a fair one. This formula is in strictness only applicable to sections symmetrical in at least three directions. Given, a strut 6×4 inches, it will evidently not fail under compression in the 6-inch direction, so that any theory of failure of the bridge founded upon this formula, is manifestly at fault as a theory, and the lesson of this accident is, that we should construct columns to which Gordon's formula will apply, or whenever the design involves an unsymmetrical column, we should stiffen it in the weak direction.

It is perfectly safe to construct iron bridges *without* cast iron bearing pieces, and to use pin joints, for struts as well as ties. The same safety,

§ 7. It shall be the duty of all railroad companies or other corporations erecting a bridge for public travel, whether by contract or otherwise, to keep on the spot a competent engineer to superintend the work, who shall have power to reject any piece of material which may have been injured, or which may be imperfect from any cause.

§ 8. All railroad bridges in this State used for public travel and having over a 15 foot span, or having a truss, shall be inspected once every month by some competent person appointed by and in the employment of the corporation owning or using the bridge, for the purpose of seeing that all iron posts are in order, and all bolts screwed home, that there are no loose rivets, that iron rails are in line and without wide joints, that the abutments and piers are in good condition, that the track rails are smooth, and that all wooden parts of the structure are sound and in proper condition, and that the bridge is safe and sound in every respect. The person so inspecting railroad bridges, shall, as often as once in two months, make report, under oath, giving a detailed statement of the condition of each bridge to the general manager or superintendent of the railroad company employing him, who shall forthwith forward the same to the Commissioner of Railroads and Telegraphs, and such inspection, in whole or in part, shall be made and reported as aforesaid oftener than once in two months, if required by said Commissioner, and it shall be the duty of said Commissioner to prescribe the form of blanks to be used by such inspectors of railroad bridges, embracing such information as said Commissioner may desire.

§ 9. All highway bridges for public travel of more than 20 foot span, in or near any city, shall be carefully inspected as often as once in 3 months, and all other of such bridges having more than 20 foot span, as often as once in 6 months, by some competent and suitable person. For city bridges, the Mayor shall appoint the inspector, and the reports shall be made to him and filed and preserved in his office for public inspection. For county bridges, the County Commissioners shall appoint the inspector, and such reports shall be made to the County Commissioners and filed and preserved in the Auditor's office for the public. For all turnpike bridges, such inspection shall be made by an inspector appointed by the company using the bridge, and report shall be made to the County Commissioners of the proper county,

applies to making available all the stiffness attainable in the tension members, and I am very certain that economy of construction can be made to accompany prudence or safety if these latter are required independent of management or contractors' interests, by government inspection.

Upon the further question of Congressional action on the subject of railway construction or traffic, I wish to say, there can be no question as to the desirability of a government inspection for the protection of travelers on railroads—I should prefer the English word, railways—and this inspection should have power to enforce results through the courts of the United States, by suspending traffic, or otherwise, as the court should direct.

The alternative ways of protecting the public are :

First.—Those of legal requirements obliging owners or directors to have inspections made previous to occupancy or use and at stated intervals of time afterwards, by their own officers or employees. This method would fail, as is evidenced by the Ashtabula accident, because, however competent or careful such officers or employees may be, their inspections would be biased in favor of the employers' interests as a matter of

and filed and kept as Reports on County Bridges. All reports shall be made under oath, and the Mayor or County Commissioners, as the case may be, shall allow the inspector such sum for his services as may be just, which shall be paid in the same manner as claims against the city or county are paid.

§ 10. All corporations operating lines of railroad in this State, shall, within sixty days from the taking effect of this act, report to the Commissioner of Railroads and Telegraphs, to be preserved in his office, a detailed description of all bridges in this State of more than 15 foot span, or having a truss, on their respective lines, and used for public travel. Said report shall be under oath, and made by some competent person in the employment of such company, and shall include the name of the stream or obstacle spanned, its location in miles from the nearest prominent point on the road, with the number or other designation of each bridge, the number and length of spans, their strength, the dimensions of all its important members, abutments and piers, with the kind of material used in the same, and the foundation, and the age of the bridge, and the date when any important changes or repairs were made in the bridge.

§ 11. The Governor shall, on the nomination of the Commissioner of Railroads and Telegraphs, with the advice and consent of the Senate, appoint some competent expert, at a salary not exceeding \$3 000 a year, who shall have cognizance of the construction and maintenance of every bridge intended for public travel in this State, and who shall hold his office for the period of five years, unless sooner dismissed by order of the Governor for reasons affecting his efficiency, in which case such reasons shall be given in writing by the Governor, and shall be entered in full upon the public records in his office. Such expert, before he shall enter upon the discharge of the duties of his office, shall pass a successful examination as to his mathematical and mechanical competency, before a committee of three Members of the American Society of Civil Engineers, and receive their endorsement in that behalf, and he shall, moreover, take and subscribe the following oath, to be administered by some officer authorized to administer oaths in this State: I, ———, do solemnly swear that I am not directly or indirectly interested in any railroad or turnpike company, or in any bridge

economy or profit. Strictures upon, or condemnation of, a bridge or piece of road bed would probably result in prompt dismissal of the offending engineer, as it did for Mr. Tomlinson in the Ashtabula case.

Second.—In popular estimation, the *insurance* method, either of the bridges or of the individuals. The railway companies or the travelling community are to be insured against accident. The policy of this proposition can be discerned by the statement that the value of the insurance to the insurers, not to the insured, lies in the proportion of the risk. It would be for the interest of the insurance companies or agents to accept anything proposed, over which a train could be run at all, rather than to examine or test for themselves at a greater expense than the small percentage of risk. In fact, the railway company and travellers insure themselves now, with very little loss of property or life. Still these accidents are preventable, certainly in some further degree than has yet been accomplished, and it becomes a great public duty to take measures to this end, and another way should be considered.

Third.—The protection of the person and property of the traveller or of goods in transit is one of the prime functions of government, and on the establishment of our Federal Union, this duty was devolved by the Constitution upon a general government, not merely in permission, but

company or bridge patent, or in the manufacture or sale of any bridge materials, and that during my continuance in the office of inspector I will not become so interested, and that I will honestly and faithfully perform, with my best skill and ability, every duty in said office, without fear, favor or affection, so help me God. Said oath shall be filed and recorded in the office of the Governor.

§ 12. Said expert shall be subject to the direction of the Commissioner of Railroads and Telegraphs, and all papers, calculations, reports, etc., pertaining to bridges, shall be filed and preserved in the office of said Commissioner. It shall be the duty of said Commissioner, whenever he is notified, or shall receive information, whether official or otherwise, or shall have reason to suspect that any railroad bridge, or any other important bridge, is defective, to immediately cause the same to be inspected by said expert, and if found unsafe, he shall prohibit its use till put in safe condition, and so pronounced by said expert.

§ 13. All railroad, turnpike, city, county and state officials, having in charge the letting or construction of any bridge or bridges of more than 35 foot span, shall submit to said expert, a strain sheet and drawings of the proposed structure before work has commenced thereon, who shall examine and certify its correctness, if correct, and make such alterations as may be necessary, if it be faulty in design, or scanty in materials according to the standard prescribed in this Act, and on the completion of such bridge, said expert shall critically examine the work in all its details, comparing and verifying the sections on the strain sheet with those of the actual structure, and if these last are insufficient, to forbid the use of the work till the bridge is made sufficiently safe and strong. A copy of all plans and strain sheets submitted to said expert shall be preserved in the office of the Commissioner of Railroads and Telegraphs, and changes made, from time to time, shall be noted on the records or files of said office.

§ 14. If said bridge, examined as mentioned in the last section, is up to the standard in all its parts, said expert shall give triplicate certificates to that effect—one to the builder, one to be filed in the office of the Commissioner of Railroads and Telegraphs, the third certificate shall be given to the railroad company; if a highway bridge, then a tablet shall be placed on a

as an obligation. Under this constitutional provision, Congress has passed laws relative to steam vessels, and established a Board of Inspectors with very ample power and authority. The results of this legislation have been evidently satisfactory in the prevention of disasters, which previously were of frequent occurrence.

The saving to the steamboat owners, if it be assumed that boiler explosions would have, without this inspection, happened as often as before the law, has been much greater in money value than the cost of inspection for any year under the law, while the public has gained almost perfect immunity from accident, except from criminal carelessness.

It is not to be supposed that the railway companies will favor any "interference." The mild proposition to establish a Board of Undertakers suggested in the bill of Mr. Garfield, will fail to meet the views of those who only wish to be let alone. I doubt if the public will be satisfied even if the military engineers who are assigned to this duty, prepare obituary notices of the unfortunate deceased passengers.

I do not question that from amongst the able educated men who constitute the corps of U. S. Engineers there will be found three members who will not only be capable but emulous to organize a Commission on

conspicuous part thereof, containing the third certificate, and also the names of the builders, the name of the officer or officers who accepted the work, the strength of the bridge, as designed, and the year of its erection. Any person who shall wilfully injure or fraudulently destroy such tablet shall be deemed guilty of a misdemeanor, and, on conviction thereof, shall be imprisoned in the county jail, not more than six nor less than three months.

§ 15. In all cases, there may be an appeal from the decision of said expert to the Commissioner of Railroads and Telegraphs, who shall carefully inquire into the matter in dispute; and any modifications of the orders or decisions of said expert, or changes ordered or made by said Commissioner shall be in writing, and spread upon the records in his office; and if said expert, from press of official business or other cause, shall require assistance, said Commissioner may, from time to time, employ such assistance as he may deem proper, and pay for the same out of his contingent fund.

§ 16. It shall be the duty of the Commissioner of Railroads and Telegraphs to stop the running of trains on all railroads in this State where the company or companies operating the same either neglect or refuse to comply with the provisions of this Act; and in case of injury to person or property, by reason of defect in any bridge in this State, this Act shall not be construed or have the effect to diminish the liability of any corporation or authority, at the time of such injury, using the same. Any person wilfully taking a false affidavit, under any of the provisions or requirements of this Act, shall, on conviction thereof, be imprisoned in the penitentiary, at hard labor, not more than ten nor less than one year; and any person stealing, concealing or suppressing, or fraudulently destroying any paper, calculation, plan, or other thing required to be kept in the office of the Commissioner of Railroads and Telegraphs, shall, on conviction thereof, be imprisoned in the penitentiary, at hard labor, not more than three years, nor less than one year.

§ 17. The standard loads on bridges of narrow gauge railways shall be 30 per cent. less than those provided in the preceding sections of this Act. The word bridge in this Act, shall be held to include trestle-work, each span of trestle being accounted as a separate bridge.

Railway Traffic and Construction, even under the dubious authority of the bill "to provide for the more thorough investigation of accidents on railroads," but their number will prevent any useful result, and their authority, or want of authority, will make the Commission almost a laughing stock. We do, the nation does, want a supervision of the railways for the safety of passengers and the prevention of accidents. A Board of Commissioners of eminent civil engineers, appointed for districts like the steamboat inspectors, would evolve in a few years such rules and regulations as would meet fully the exigencies of the case. The present moment is an admirable one for commencing such a Board. Public opinion is fully awake to its necessity, and only calls for direction in the right path to insure both public and legislative success. With this conviction, I would suggest that one of the steps most evidently proper, is the expression of opinion of our Society, as a society, upon the subject, to be used before Congress at the coming extra session. I think some members of the proper committee of the House or Senate will gladly take up the question, if it be offered in a practical form from the American Society of Civil Engineers.*

MR. THEODORE COOPER.†—I am glad to find that Mr. Macdonald reports the material of this bridge, as of a good character, for had it been of a poor quality there would be greater difficulty to make those who have charge of iron bridges believe that there can be any other reasons for iron bridges failing.

I would class the "ills that bridges are heir to" under the following general heads:—the material; the design; the manufacture of the parts; the erection or assembling together of the parts, and finally, the general neglect or abuse they afterwards receive from the hands of those who have the immediate care of them.

1°.—THE MATERIAL.—The necessity for employment of good material is more fully appreciated, I think, than the other points in reference to which the bridge may be weak—and certainly far more stress has been paid to this part of the subject.

2°.—THE DESIGN.—By faults of design I would not only include those due to the general form or style of bridge and improper determinations of the strain sheet, but those which might more properly be classed as faults of guesswork. In too many of our built structures, it would be found, that the so-called minor details of them have been proportioned

* This was presented March 6th, 1877. † Presented April 4, 1877.

in this manner. To illustrate, I could point out two sets of plate girders of exactly the same general proportions, and carrying the same load, in which the rivets have been spaced by guesswork in one and by a determination of the "strains transferred" in the other. The web sheet in the first, has a strain in each rivet hole at the ends of the girder equal to 40 000 pounds per square inch; while in the other, the strain upon even the rivet holes is less than 12 000 pounds per square inch. Now the former case is a very frequent one. Do not understand, that such faults are confined to riveted work, for were it necessary to multiply illustrations, I could show similar neglect in bridges with pin joints. Now to a superficial examination, the above girders are of equal strength, and should one set fail and not the other, the explanation would undoubtedly be sought in the "crystallization of the metal," or some other equally mysterious reason, instead of laying the fault to guesswork designing.

3°.—THE MANUFACTURE.—However carefully designed a structure may be, if the workmanship is of a poor or defective character, the parts are not in a condition to perform their proper duty. Incorrect lengths, poor joints, bad riveting and welded connections are only a few of the faults that can be laid to the manufacture. There is one class of faults which should, perhaps, be divided with the design; that is, such forms in the original design as cannot be properly made.

4°.—THE ERECTION.—A bridge that has been designed and manufactured properly, may afterwards be so badly erected that its parts are not in a proper condition to do their duty. If the workmen are allowed to change the lengths and fittings, by chipping and filing of pinholes and abutting surfaces, by gouging the rivet holes, heating and drawing out bars, and other improper practices, to "make things come together," all previous care has been labor lost. Also by carelessness in marking, or want of proper instruction, pieces may be put into the wrong position.

5°.—THE TREATMENT AFTER COMPLETION.—Iron bridges—being structures formed of numerous parts held together by pins, rivets and friction of surfaces—require a certain amount of care to keep all parts in a proper condition. Nuts will work loose, rivets break, &c., minor points which may by neglect lead to greater troubles. The best of iron bridges need some care, and when we come to bridges which have faults of design or workmanship, a much more careful watch is needed over the

weak or doubtful points. Many bridges have adjustable pieces upon the proper condition of which depends the adjustment of the bridge. Upon the proper condition of the counter-rods especially, does, the proper action of the trusses depend. Unfortunately no method of fixing the lengths of these rods unalterably, after the bridge is adjusted, has been adopted; and equally unfortunate is it, that those who have charge of bridges imagine the panacea for all ills of bridges is to screw up the counter rods, thus introducing internal strains into the structure, relieving some parts from strain and increasing the strain upon others. In some particular styles of bridges, this is an especial source of danger.

I have noted the above points to illustrate the fact, that the strength and condition of our bridges cannot be determined by overhauling the strain sheets solely. Their weak points should be determined by a thorough examination in reference to all the foregoing heads.

I think much error upon the subject of the safety of bridges arises from the false idea in regard to the factor of safety. It is generally claimed, that our bridges have a factor of safety from 4 to 6, whereas they have more generally a factor of safety of $1\frac{1}{2}$ to 2. A confusion has been made between the factor of safety of the material and that of the bridge. A skeleton structure of any kind is formed of pieces combined to act together in support of a load; the strain upon each piece is dependent upon its length and position relative to the others, and all the parts are assumed to elongate and compress proportionately to the strain coming upon each. Should any one piece take a permanent elongation, the condition of the strains is changed, and they necessarily must be excessive upon certain parts. Now, for a practical material, I do not think we can claim over 20 000 to 22 000 pounds per square inch, as the limit of elasticity of wrought-iron. This, therefore, is the limiting strain to which our structures should be considered as confined, and to which we must refer for the factor of safety of built structures.

Most of our iron railroad bridges are computed to have a strain of 10 000 pounds per square inch upon their parts when under their assumed maximum load, which is usually taken as the greatest load from the rolling stock in use upon the road, equally distributed upon both rails, but does not usually allow for the additional strains produced by this same load passing at a high speed, and subject to oscillations and shocks from imperfections of the track and rolling stock. Considered, then, under practical conditions, many of our bridges undoubtedly have

strains exceeding 10 000 pounds per square inch at times ; so that a factor of safety of $1\frac{1}{2}$ to 2 is all that should be claimed for them. By appreciating the smallness of the factor of safety, the necessity for keeping them in good order would become more apparent to all in charge of such structures. Too many of those having charge of bridges, and some of those who built them, seem to have great faith that the bridges will carry five times their greatest assumed load, and therefore they have a large margin for neglect and abuse. If iron bridges are to be what they are supposed to be, and what they could be under proper management—permanent structures—a better idea in reference to their powers must be had. Because they do not break down after a dozen years' use is no evidence that they are safe or that they are in tolerably good condition.

To prevent further repetition of such accidents as occurred at Ashtabula, the exact character of every bridge, in reference to all the points mentioned, should be determined and the proper remedies applied before it is too late. A proper inspection must include the smallest details upon which the strength of the structure depends. Bridges cannot be inspected from a railroad train, nor can the faults be determined by running engines over the structure to see if it will fall. Each individual part of the structure must be examined, and the manner in which each part acts under the rolling loads. If it is found that some one part does not take its proper strain, it must be determined which other parts are taking the extra strains necessarily developed. The person who makes the inspection should be qualified to judge of each of the points before mentioned ; the material, the design, the workmanship of manufacture and erection, and finally, whether the structure is actually in the condition called for by theoretical results, or have certain parts more work to do than they should, through improper adjustments.

MR. C. SHALER SMITH.—It is doubtful whether any of the hypotheses hitherto advanced will fully account for the fall of this bridge. In estimating the strain on the lug of the angle-block, Mr. Macdonald has omitted the friction, which, if the rear driver of the leading engine was directly over the second angle-block at the moment of failure, was $197\ 880 \times 0.175 \times 2 = 69\ 258$ pounds, and the direct pressure to be transmitted by each lug was $\frac{110\ 600 - 69\ 258}{2} = 20\ 671$ pounds, or less than 4 000 pounds per square inch of shearing section, after the deduction is made for the air-hole in the casting. Moreover, had this lug given way

before the fall of the truss, the angle-block would have turned on the counter-brace as an axis, the entire strain would have been transferred to the three inner braces of that panel; these would have been compressed along their inner edges, and failure would have resulted from their lateral flexure *outwards*, while the remaining lug would have been twisted off by the turning of the angle-block under this action of the adjacent members. The other lug appears to have been unbroken, however, and Mr. Philbrick's sketch* shows that the braces doubled inwardly, not outward. This, with the fact that the chord members of that panel show lateral bending only, is clear evidence that the braces were bent, and probably the lug broken, during the fall of the bridge.

From the sketch referred to, it appears certain that the three eastern panels fell to the south, as well as joint 12 at the southwest end, the rest of the bridge falling north. This shows that the failure was not due to local crippling, but to some cause which effected the entire structure, and indicates that the lateral and vertical connections are the points to be considered. But all points connected by vertical laterals fell in pairs and in the same direction, therefore these were not in fault. The examination is now narrowed down to the horizontal laterals or wind braces. Here we find that all points connected by this system fell together, except joints 3 and 5, in which case 5 fell to the north and 3 to the south, showing that the wind bracing connecting them was broken prior to the fall, so as to admit of the joints moving in opposite directions. This gives a clue to the mode of failure. The train being on the south truss, deflected it below the truss on the north (which bore but 14 per cent. of the load), and consequently all the vertical laterals in the joints, from 3 to 11, were loose, leaving the top chord of the south truss to be held in place by the horizontal rods only. A wind pressure of 18 pounds per square foot on truss and train would break the splice in the rod leading from 1 to 3, which Mr. Philbrick describes, and the parting of the rod from 3 to 5 would immediately follow. Joint 3, would move outwards, propagating a lateral bending wave along the chord, which would continue until a point was reached where the vertical laterals were taut. This point was joint 13, where the deflection was a minimum and the "*snap*" of the wave would come at the nearest unprotected point to joint 13,—to wit, joint 12, where there were neither vertical or horizontal rods to hold it in place. At the moment of failure, joint 12, being the

* Plate XI, page 86.

middle of the wave, would move outwards, while the joints on either side would move in the opposite direction. This action would cause the end angle-block, 13, south chord, to twist to the northeast, and the main braces would slide off it sideways to the north, leaving the angle-block free to be driven down the vertical ties, by the live load, during the fall. During this action of the top chord, the second set of braces would double inwardly, just as shown in the sketch. The "snap" at the end of a wave action, caused by lateral flexure, will be readily understood by any member who had occasion to destroy a bridge during the late war, and who noted its mode of falling, according to the part of the truss selected for destruction.

One fact to be especially noticed, in connection with this accident, is the exactness with which the behavior of the material has sustained the correctness of the modern forms of the Rankine formula. When the structure was first erected, the flanges of the braces were placed in a vertical position. With the striking of the blocking, came the extension of the lower chord, the compression of the upper chord and the consequent motion of the angle-blocks, in top and bottom chords, away from each other. If the angle-blocks were square to the braces before the span was swung, they certainly were not so afterwards, and the beams were therefore compressed on opposite edges of their flanges by the upper and lower angle-blocks, against which they rested. The formula for an I beam strut, loaded diagonally on the flanges, is

$$\frac{38\,500}{1 + \left(\frac{l^2}{6\,130 \times r^2} \right)} = S,$$

and substituting the values of l^2 and r^2 for the strut in question, we have

$$\frac{38\,500}{1 + \left(\frac{67\,500}{6\,130 \times 0.835} \right)} = 2\,711 \text{ pounds per square inch ;}$$

or for each beam, $2\,711 \times 9.6 = 26\,025$ pounds.

It appears, however, that they *did* fail, when the span was first swung off, with about 24 500 pounds per beam. When they were turned, so that their webs were vertical, and the flanges presumably square to the angle-blocks, the formula became

$$S = \frac{38\,500}{1 + \left(\frac{67\,500}{25\,000 \times 0.835} \right)} = 9\,100 \text{ pounds per square inch.}$$

Flexure of the chord or turning of the angle-blocks, so that one half of the braces in any of the four end panels would receive the entire load, would at once produce failure, as the ordinary strain on these beams, when the load was equally divided, was about 4 600 pounds per square inch when the drivers were immediately over the panel under consideration. In panel 13, of the top chord, there was a similar state of things. The three I beams, which took the strain of the main braces, were separated from each other by two beams, parallel with and between them, but which had no steadying longitudinal strain upon them whatever. The middle beam of the three was so held, that the formula gives it a value of 21 000 pounds per square inch; but the two outer beams were 22 feet between adequate lateral supports, and consequently their strength did not exceed that of the braces, or 9 100 pounds per square inch. A lateral flexure of $1\frac{1}{2}$ inches at joint 12, would produce a strain of over 10 000 pounds per square inch in one or the other of the outer beams, according to the direction of the flexure, and failure at the joint would inevitably result. In the other chord panels, all the members were under strain, and there were no loose, unstrained beams lying between and separating the working members, so it is probable that elsewhere the three inner chord beams were all nearly up to their theoretical value of 21 000 pounds per square inch.

In considering both chords and braces, the effects of the floor beams on the one and counter-brace clamps on the other have been left out, as, their action being uncertain, they have no place in rigid theory.

To conclude this part of the subject, Mr. Clarke's assertion,* that any iron bridge expert would have "condemned the structure on sight," is most emphatically correct. An inspection by an engineer of this class would have been followed by riveting cover plates of boiler iron on top of both chords and braces, and the addition of another system of lateral bracing, when the Ashtabula bridge would have been good for half a century to come.

Mr. Macdonald does scant justice to the iron bridge building of former times, in his remarks† concerning the extent of knowledge on the subject twelve years since. Prior to the war—from Louisville, Baltimore and Richmond, as centres—a large number of iron bridges were constructed in the States of Kentucky, Maryland, Virginia, North Carolina, Tennessee and Mississippi. Many of these were important works

* Page 87. † Page 83.

and of excellent design. Excepting the material in compression, which is cast-iron, I know of no better bridge in the country to-day than the one over the Elizabeth River at Norfolk, Va. Following these, many more were built along the Illinois Central and other Western railroads by the Detroit Bridge & Iron Works; while at the time that the Ash-tabula bridge was under consideration, Mr. Linville had already finished his magnificent span at Stenbenville, and the designs for several of the great bridges of the West had taken form and shape. Mr. Macdonald's view of the case is correct so far as New York and New England are concerned, but his remarks are not applicable to the border states, Pennsylvania and the Northwest.

Concerning the lesson to be learned from this particular bridge failure, in a mechanical sense I see none, whatever. The construction of the truss violated every canon of our standard practice, and its failure has taught us nothing that we did not know before, except, perhaps, that it is possible for a bridge with a factor of safety of $1\frac{1}{2}$ to last for eleven years.

While on this part of the subject, I desire to call attention to a dangerous practice on the part of railroad companies, *i. e.*, that of having their iron bridges under the charge of the Road Department. The "Master of Bridges and Buildings" does very well so long as the railroad is furnished with Howe trusses; but when these are replaced by iron, they should be under the charge of men who understand them, and who report to the chief engineer only. In addition to this, the bridges should be inspected twice a year, as Mr. Macdonald suggests, by an outside expert of known ability and experience.

Concerning the law proposed by Messrs. Adams and Garfield,* it is an excellent one, and is probably as far as the United States government can constitutionally go in this direction. Railroads are not navigable waters, and consequently not under the direct jurisdiction of the national legislature. A mandatory law, such as has been suggested by Mr. Clarke,† would have to be enacted by the several State governments. I should amend Mr. Garfield's law, however, by changing the composition of the Board to one Engineer Officer, one Civil Engineer who should be a known expert, and one officer from the Ordnance Corps.

In concluding this subject, permit me to call the attention of the Society to the fact that, in 1874, Capt. Eads, Col. Flad, and myself

* Page 202. † Pages 87, 207.

foresaw the present state of affairs; and in the report published in the Transactions for May, 1875,* strenuously urged that a law should be drafted at once covering the question of "bridge accidents," and during the following winter pressed through the various State legislatures by the individual members. Had this been done at that time, it is more than probable—as our Ohio members are both active and influential—that the Ashtabula bridge would have been condemned and made safe, and the horrors of that fatal night averted.

Finally, to fix conclusively where the responsibility of the policy of non-action in this matter is to be borne, I move the adoption of the following resolution:

Resolved—that a committee of five, whose names shall be selected by letter ballot, shall be appointed to draft a law covering the points outlined on pages 125, 126, 127 and 128, of Transactions, May, 1875, adding thereto the necessary provisions to secure the inspection by experts of all questionable bridges now in existence.

And further, that this law, so drafted, shall be submitted, together with a resolution recommending its adoption by the various State legislatures, to this Society for letter ballot; and, if approved, that printed copies of the said law and the accompanying resolution be sent to the members of the Society, with a request "that they move actively—each in his own State—towards procuring the passage of the specified law by the various State legislatures during the coming winter.†

This action on the part of the Society will in no wise clash with the Garfield Commission. It will simply cover ground that the latter, owing to want of constitutional authority, cannot do, and will pave the way for the suppression of existing man-traps, numbers of which are doubtless known to the experts among our members.

I cannot conclude without expressing my thanks to Messrs. Macdonald and Philbrick for the public and professional spirit they have shown in this matter. The sketch contributed by Mr. Philbrick is exceedingly valuable in this connection.

* Vol. IV, page 122 and following.

† The resolutions were adopted by the Convention, and under By-Laws, Section 30, referred to the Board of Direction, for submission to the Society.

CLXIII.

IMPROVEMENT OF ENTRANCE TO GALVESTON HARBOR.

A Paper by CHARLES W. HOWELL, Corps of Engineers, U. S. A.,
Member of the Society.

READ AT THE NINTH ANNUAL CONVENTION, APRIL 24TH, 1877,
CALEB G. FORSHEY in the Chair.

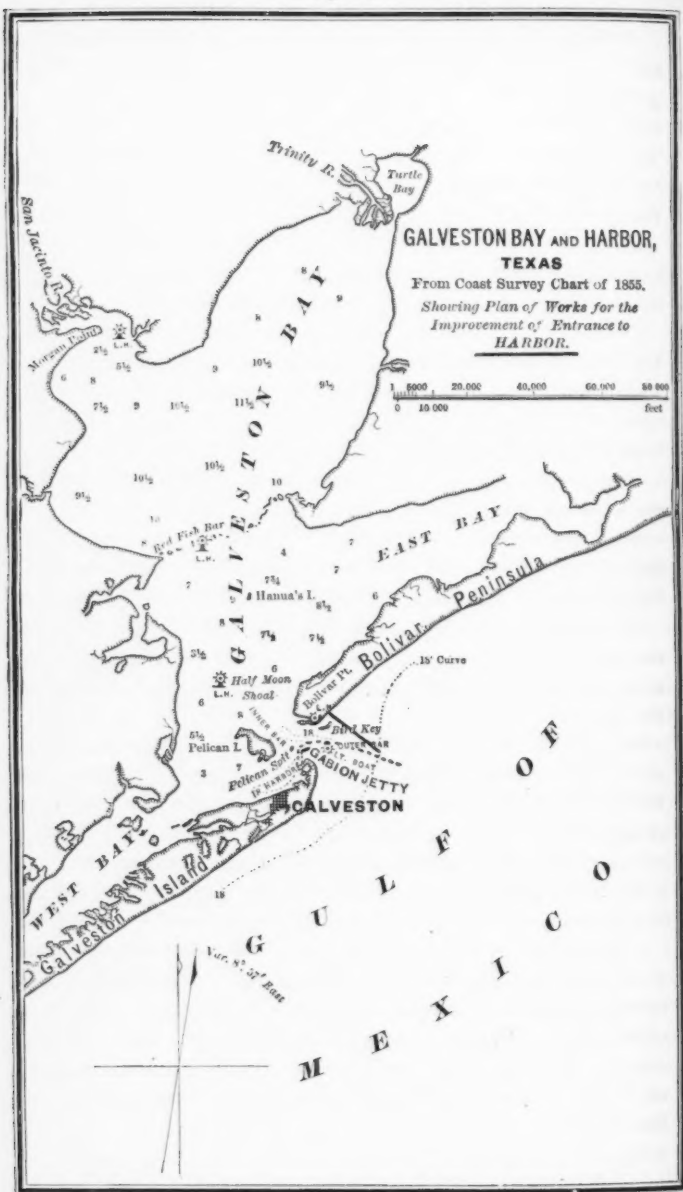
The work now in progress for the improvement of the entrance to the harbor of Galveston, Texas, consists in the application of the jetty system, in one of its well-known forms, to a drift bar.

The jetties are of the type known as "right line," "drowned jetties," and are designed simply to arrest, in a measure, the sand drifted along the gulf coast by the action of the winds and waves, and to train the currents caused by the daily tidal discharge of Galveston bay. The most interesting feature of the work is esteemed to be, the novel method adopted for the construction of the jetties or "gabionades," as they have been styled.

I will confine this paper mainly to a description of this method, first, however, giving a general idea of the conditions calling for improvement of the harbor and those which have governed in the selection of a plan for improvement.

Galveston is the principal seaport of the rapidly growing State of Texas. It occupies a central position on the coast, the more populous portions of the State considered, and with this position is connected both by rail and water. Nature has given it the best outlet channel to be found on our coast, west of the Mississippi river, a channel ranging from 11 to 12 feet in depth at mean low tide. This depth, of late years, has been found insufficient to meet the necessities of commerce. In consequence, the present work was undertaken by the general government, with the reasonable expectation of increasing the channel depth, to at least 20 feet.

Fig. 35.



The city of Galveston is located on the east end of a long, narrow, sand island, running parallel to the main land and separated from it by a shallow bay called "West bay." North of the east end of the island is the broad expanse of "Galveston bay," and to the northeast is "East Bay," running well up toward Sabine pass and behind Bolivar peninsula. These bays, having a combined area of about 455 square miles, are connected with the gulf by the Bolivar road-stead, separating Galveston island from the Bolivar peninsula. Through this, the tides daily flow and ebb, creating tidal currents which have made and which maintain a broad deep channel between the headlands.

This channel is separated from the deep waters of the gulf, by a sand bar of nearly semi-circular shape, formed in accordance with the well-known laws governing the formation of purely drift and tidal bars. This is called the "outer bar," and the normal depth of channel across it has been, for such time as we have record, about 12 feet.

Galveston harbor is a branch of the Bolivar road-stead, leading along the inner face of Galveston island toward West bay, and at the time of commencement of the work which is the subject of this paper, was separated from the road-stead by a bar, across which there was a channel but 11 feet in depth. This is called the "inner bar."

The channel across this, is reported to have been 30 feet in depth in the early part of this century, and within the time covered by authentic record is known to have steadily shoaled up to the time of commencing the work of improvement. This shoaling, as it now appears, was rightly attributed to the continued abrasion of the east end of Galveston island, and there was reason to fear that if further abrasion was not prevented, the harbor would, at no very distant day, be rendered useless. To simply prevent further abrasion would not insure greater depth of channel, it was, therefore, decided, as the first step, to build the end of the island out to the place it was said to have held when the channel was 30 feet in depth.

This work was commenced by the city of Galveston before the assistance of the general government was obtained, and a pile pier extended several hundred feet beyond the end of the island, but not far enough to obtain valuable results. From the outer end of this pier, which was partially destroyed by the great cyclone of September, 1875, and has since been repaired, a gabionade was carried out 1200 feet, which had the effect of deepening the inner bar channel to $16\frac{1}{2}$ feet, mean low tide, and of shortening the bar, between 18 foot curves, to 450 feet, a

shortening of 5 240 feet. This condition has been maintained without material change for a year and a half.

Further extension is now in progress, and further improvement of channel is anticipated. On the gulf side of the gabionade, the changes effected by it are no less marked than those effected in the inner bar channel. The shoals lying immediately between this and the gulf to the south and east have been largely built up and extended, and have served to divert the ebb-tide currents more directly along the axis of the roadstead. In this way, a direct broad and somewhat deeper channel has been produced across the outer bar, and that on the line designated in the general plan for the channel after completion of the works proposed.

The work designed for improvement of the outer bar is to consist of a pile-pier extending directly out from a point near the end of Bolivar peninsula to the curve of 6-foot depth, after which it is to be prolonged as much farther as may be found necessary, by a gabionade.

The gabionade is simply designed to form the nucleus for a sand shoal, which latter is to perform the office of guiding the ebb currents and of protecting the results of their scour from being entirely neutralized by shore drift. Entire protection is not to be sought, for it is not now believed that it will be necessary.

The work planned for the outer bar, is to run out from the Bolivar peninsula, instead of making an extension of the island gabionade toward the gulf, for the reason that the prevailing winds are from an easterly direction, and the consequent race of the waves along the coast, from which protection is most desirable, is from that direction.

The extension of the island gabionade, by turning it so as to bring its outer bar portion parallel to that from the peninsula, has been considered as a possible necessity, but will not be undertaken until the latter is completed and shown to not give a desirable measure of improvement. If fully completed as laid down on the chart, (Fig. 35,) the peninsula gabionade will be about $4\frac{1}{2}$ miles in length, and laid down in water ranging from 4 to 12 feet in depth. There is some reason to think, however, that it may not be necessary to project it out more than $2\frac{1}{2}$ or 3 miles, and a large portion of this it is expected will be so far advanced this year as to give marked results,—perhaps as good as those obtained on the inner bar.

It is also thought, that it will not be necessary to build the gabionade up as near the surface of the water as the plane of mean low tide.

These, however, are but impressions, which the work will be so conducted as to test.

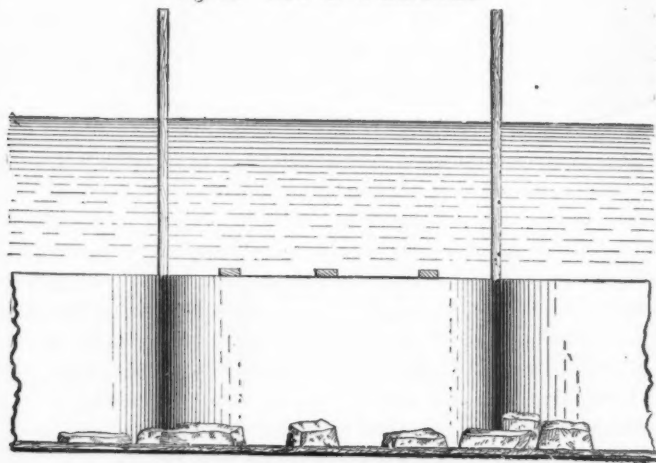
Tidal currents being the active agents upon which the work depends for success, it is proper to state that their velocity is so variable, they can seldom be depended upon to move more than the fine, rounded sand, of which the bars are mainly composed, to a depth of 24 feet. From hourly observation made previous to commencement of the work, and for ten consecutive months, it was ascertained that the extreme range of the tides for that period, during which no extraordinary storms occurred, was $5\frac{1}{2}$ feet.

The plane of mean low tide established from this, and since retained as our plane of reference, proved to be that of the lowest high tides for that time. The tides ranged above this, $3\frac{1}{2}$ feet, and below it, $2\frac{1}{2}$ feet. The greatest daily range was 3.2 feet, and the least 0.4 feet.

The ebb tides are considerably re-inforced by the drainage of the basin tributary to the bays; this, however, is only notable during the rainy season. The drainage is ponded back in the upper portion of the bay by the flood tides, and there deposits its sedimentary matter, after which it passes to the gulf with the ebb, clear, except when the bottom of the bay is stirred by storms.

The outline chart, Fig. 35, shows the general features of the bay, harbor and entrance, and the location of the gabionades.

Fig. 36. VIEW OF A GABIONADE.



A gabionade consists of a number of gabions, filled with sand and sunk in line upon fascine or hurdle mats, with which the bar surface along the line is carpeted to prevent settlement of the structure. The gabions used on this work are large boxes with semi-cylindrical ends and straight sides of basket-work, and with flat tops and bottoms of heavy planking. For the basket-work, we have used small pine poles as stakes and sea cane for wattling. The stakes are from $1\frac{1}{4}$ to $1\frac{1}{2}$ inches in diameter, and are fastened to tops and bottoms through auger holes in these. The sea cane averages about $\frac{1}{2}$ inch in diameter at butt. The cane is thought to be not subject to attack of the teredo—it is also enduring under water, and very pliable for several weeks after cutting. It is abundant and cheap, as is also the case with the pine stakes.

Fig. 37. VIEW OF GABION.

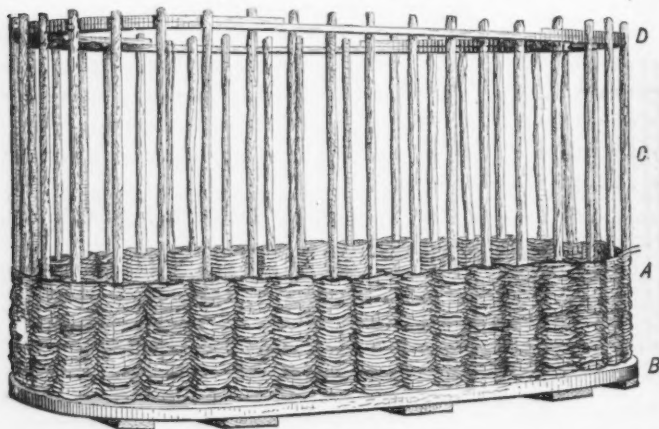


Fig. 37 shows wattling of sides and ends; *A*, being wattling—*B*, bottom—*C*, stakes, and *D*, temporary frame.

The basket-work, both inside and outside, is heavily coated with a rich hydraulic cement mortar, making the walls from 5 to 6 inches in thickness, and this is allowed to weather-season for three months before being transferred from the construction yard to the gabionade. The tops are carbolized and covered with mortar (the same as the walls), which is secured in place by bevelled cross-pieces on the top.

A stiffening frame is provided inside the gabion, at 2 feet from the bottom, and another at 2 feet from the top. Two holes are left in the top for the introduction of sand, and are closed after filling.

The longer axis of each gabion is 12 feet, the shorter 6 feet, and the height is 6 feet. The weight filled, is about 19 000 pounds; smaller sizes have been used at special points, for spurs.

Of the mats, for preventing settlement of gabions and for modifying scour in advance of construction, two kinds are in use.

The "fascine" mat is made up of fascines, each 12 feet long and 6 inches diameter, fastened together with wire to form mats 12×6 and 12×9 feet. One of the former and two of the latter are used for each gabion. The fascines are made up from the cane trimmings and from cane unfit for wattling. The "hurdle mat" is made with pine poles and a wattling of untrimmed cane, making a bushy mat about 4 inches in thickness, 12 feet in width and 24 feet in length. Both kinds of mats are strong and flexible enough to conform to the shape of the bottom.

Fig. 38. PLAN OF GABION AND FASCINE MAT.

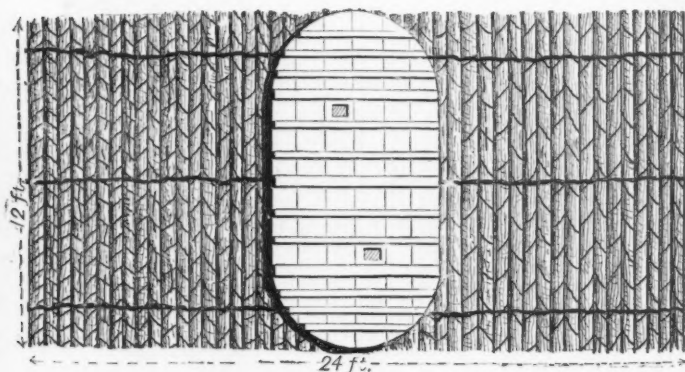
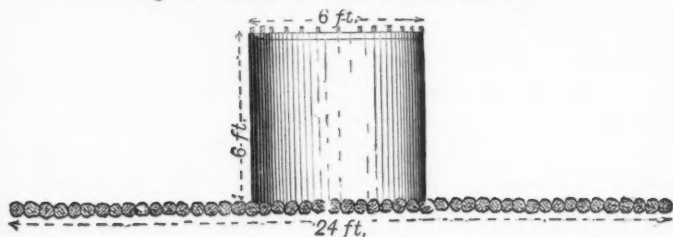


Fig. 39. ELEVATION OF GABION AND MAT.



The mats are floated to positions and sunk by placing concrete blocks upon them. These blocks are made of broken brick, gravel and beach

sand, with only sufficient cement to bind the mass together and permit handling without breaking.

To guide in placing mats and gabions in position, the line of gabionade is marked out some distance in advance, by pine piles 12 feet apart. Between these the mats are sunk.

To sink a gabion, it is floated in between two piles, pumped full of water and lowered to its place. With a sand pump (Andrew's patent), it is then filled with sand. After completely filled, the holes in the top are closed and the gabion becomes, to all intents and purposes, a solid block of stone. Where intervals may be left between gabions, these are closed by large fascines wedged in and fastened with wire and blocks.

A description of the plant employed on the work would make this paper too long, and besides, possess no points of special interest.

In conclusion, I will remark that the work, when inaugurated, had many features considered experimental in character. Many of these have stood the test of severe trial, with credit. It was not known, if the gabions would stand against the shock of the waves during severe storms. The cyclone of September, 1875, gave strong evidence of their stability. Those in the gabionade retained their places, and even those which were in the construction yard on the end of the Island, and which were also well seasoned, although the waves broke several feet over them, at the close of the storm were found to have moved but little, to have remained upright, to have sustained but unimportant damage and to have been filled with sand by the waves breaking over them. They have been found to serve admirably as the nucleus for a sand shoal, and in but few cases has the sand failed to completely cover them so that they could only be found by probing.

The cost per running foot of gabionade, 6 feet high and 6 feet wide, was found before the storm, to be \$5.21. Since, many additions have been made to the original design, which will probably bring the cost up to about \$8.00. No exact estimate, however, will be attempted until the close of this year.

The work is claimed to have shown efficiency, durability and cheapness combined. Should further and just expectation be realized, a valuable precedent will be established for the improvement of other drift bars upon the gulf and lower Atlantic coast, where works will not be called upon to resist the action of ice.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—The Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

DISCUSSION OF SUBJECTS PRESENTED AT THE NINTH ANNUAL CONVENTION.*

ON THE FAILURE OF THE ASHTABULA BRIDGE.†

MR. W. MILNOR ROBERTS‡:—One of the important lessons to be learned from this bridge accident, and from every serious railroad disaster, is that it is to the interest of companies, as well as of the general public, that only experienced, competent men should be employed—not only in locating, planning and constructing, but in superintending the road and its structures, and in managing the running department.

The precise immediate cause, or causes, of the terrible accident at the Ashtabula bridge may or may not ever be clearly defined. That there was faulty construction, judged by the bridge experience of the present day, Mr. Macdonald has shown. On the other hand, eleven years of very heavy and annually increasing heavy use of the bridge might imply that the bridge was sufficient for the business when it was built, and that there may have been some particular cause, independently of any fault of construction, without which the bridge might possibly have continued to carry such heavy traffic safely for some years longer.

Mr. Charles Hilton has stated § that there is a railway station only about 1 200 feet west of the bridge, and that the locomotive engineers were in the habit of beginning to apply the brakes, on the westward bound trains, near to and on this bridge. Such a circumstance, in this connection, is certainly deserving of special attention, as offering a possible clue to one of the immediate causes of the destruction of the bridge. A bridge (with or

*Continued from page 222. †Referring to—CXXXVII, The Failure of the Ashtabula Bridge. C. Macdonald. Page 74. ‡Presented April 5th, 1877. §Page 206.

without a comparatively weak spot in its original construction), subject to frequent, unusual, extraordinary and irregular strains, not provided for, would of course be more liable to injury and to ultimate destruction in consequence.

The tendency of the discussion appeared to show that the bridge had a weak spot, but refused to break in that place, and broke where, according to the calculations, it may have been strong enough to stand the usual tests of the regular transit of trains. In the absence of knowledge as to the immediate cause of the disaster, there seems to be here an element of uncertainty running into the scientific treatment of the question.

Railroad companies are, and should of course feel, deeply interested in securing every safeguard against accidents ; for, next to the unfortunate individual sufferers and their families, the companies, pecuniarily and otherwise, are most interested in preventing such ; but, in the few remarks now to be made, it is not my purpose to criticise the officers or employees who may have been regarded as the custodians of the Ashtabula bridge. Nevertheless, having been invited to consider "the most important lessons to be learned from the event," I am willing to add my mite in that direction, based upon long and varied experience.

Accidents, serious accidents, will continue to occur in the use of railroads, on bridges and elsewhere, even where apparently all proper provision has been made to guard against them. Trains may become deranged, from broken wheels or axles, or derailment, on bridges strong enough for regular use, yet which may not be sufficient to withstand the extraordinary, sudden shock. It is well known, also, that during the past eleven years, the weight of locomotives and the weight of their trains have been gradually increasing on all our first-class roads, and it has been and is the duty of those in charge of such roads to see that their bridges are correspondingly strengthened. It is folly to fancy that a 25-ton locomotive, drawing a train of 25 cars at the rate of 10 miles an hour, tries a bridge like a locomotive of 40 tons, drawing a train of 40 cars at 15 miles an hour, and yet some such change as this rough example would indicate, has been going on in the use of our railroads, and it is to be feared, without, in all cases due attention to the fact.

There is hardly an excuse now for building an iron or steel bridge of inadequate strength ; that is to say, of less than the strength computed, and which it is designed to bear, and it would seem that there have

been enough accidents to prove the importance of having such bridges planned and built by bridge experts—by men whose lives and talents are devoted especially to this particular sort of structure, and whose duty it is to know that all the parts are properly proportioned and of the best material, and whose interest it assuredly is to put up structures which will not fail. I think that this is one of the important lessons to be learned.

In regard to the general subject of railroad accidents, and especially touching the Bill* presented by Hon. James A. Garfield, in the House of Representatives, January 31st, it is difficult to say in advance precisely what effect the action of such a commission would have in connection with railroad accidents, for or against them.

The Commission is to "inquire" into the number, causes and means of prevention of accidents. The first duty is merely clerical; the second involves an indefinite series of difficult and complicated investigations, scattered over 80 000 miles of railroads; the third "to inquire into the means of prevention of accidents," implies or should imply vast practical knowledge and long experience in the construction and management of railroads and of their thousand appliances.

Next, the three Commissioners are to investigate such accidents on railroads as may, in their judgment, be accompanied by "circumstances of an unusual or unexplained character." They are also to submit special reports, and at the close of each year, a general report, upon the subject of accidents—all to be submitted to the Secretary of the Treasury and to Congress.

The Bill, as it stands, is to establish a *Bureau of Railroad Accidents*; partly statistical and partly for the collection and discussion of causes and of means of prevention; "the Commissioners to be Engineer Officers of the Army." By implication—as these Commissioners are to receive compensation for actual travel and other necessary expenses—they are expected to visit in person the scenes of accidents, after they have learned of their occurrence and, as the number of railroad accidents in the United States in a single month is fearful, constant travel on their part would only permit them to attend as a coroner's jury upon a few.

Members of this Society can only offer their individual views upon the merits of this Bill. It is an experiment, and possibly, but not proba-

* Page 202.

bly, it might drift into something of practical utility, provided the Commissioners should be experienced in railroad affairs. But I fail to trace any natural connection between railroads, *per se*, and United States military engineers; which I say with great respect for United States engineers in connection with their legitimate duties—gentlemen who, probably, would not desire such an anomalous duty.

The Bill makes provision for the gathering of information and the formation, if possible, of views of the Commission, but fails to provide for any practical application, and it is at least questionable whether any advantageous practical results could ever be reached through such a Commission.

State legislatures, and Congress, from whom railroad companies may have derived their powers, have certain authority in the premises, which the companies must respect, but a state or national quasi-management of the running of corporate railroads, will probably never be fairly practicable. Companies must be held responsible by the public, and legislatures and Congress must aid the public in holding them to their responsibility, but this is not likely to be aided by the intervention of a state or national officer between the company and the public. One might as well expect to see the movement of a watch improved by the intervention of a hair among the wheels.

One good working clerk, authorized to receive the railroad publications and a few daily journals, could collate the main features of railroad accidents, and arrange them in the form of a report, that would be quite as useful as the one indicated, and at less cost. The truth is, that railroad accidents will not be prevented by any such system. Many will continue to occur, because they are inherent in any railroad system run by human agencies. Engineers, and railroad companies, can only point out and apply, measurably, safe methods, by insisting upon first class materials, first class workmen, first class superintendence and first class management, and paying therefor a fair price. The less Congress has to do with the management of running railroads, probably the better it will be for the people who use the roads.

It is a truth that railroads are liable to accidents which human prescience cannot foresee, but it is also true, that many accidents are prevented by carefully improved management. Eternal vigilance, on the part of railroad managers, is the price of safety.

CLXIV.

RELATIVE

QUANTITIES OF MATERIAL IN BRIDGES

OF DIFFERENT KINDS, OF VARIOUS HEIGHTS.

A Paper by CHARLES E. EMERY, M. E., Member of the Society.

READ AT THE NINTH ANNUAL CONVENTION, APRIL 25TH, 1877.

CALEB G. FORSHEY in the Chair.

§1. The papers and discussions on arch and continuous girder bridges, published in Transactions during the past few years, induced the writer to undertake for his own information, an investigation of the general subject, and although other matters have prevented its being carried out as yet to the extent desired, it is believed that some of the results already obtained, and the distinguishing features of the method adopted, will be of special interest to the profession.

§2. It was early seen that the investigation would be incomplete without ascertaining the proportions for each kind of bridge, so that the best examples of each type could be compared one with another. Increasing

the height of a girder decreases the moments, and consequently the chord strains and sections, while the web members are lengthened and strains on diagonals modified. It became necessary, therefore, at the outset, to ascertain the proper height for each type to secure the minimum amount of material.

§3. It is pointed out in all works on the subject, that the best angle of a diagonal, to secure economy of material, is 45° , and it is believed an opinion is prevalent that this is substantially the best angle for all cases. It has been hinted in several publications that, in a truss, the angle should be modified somewhat, when the other members are considered. In the work of Prof. De Volson Wood on Bridges and Roofs* (page 133), some analytical investigations are made on the subject, which are valuable chiefly as exercises, not being founded on practical conditions. The method of the graduate, Mr. W. Bouton, therein mentioned, takes account of all the members on the basis of a uniform load and of a uniform strain per unit of section, for material both in tension and compression. On that basis, it is shown that the vertical angle of the diagonals should decrease with the number of panels.

§4. In a valuable paper,† comparing continuous and ordinary girders, in Transactions, May, 1876, the fact is recognized that continuous girders need not be as high as ordinary girders—27 feet being assumed as the height of an ordinary girder of 200 feet span, and 25 feet that of one continuous span of same length. It is, however, stated in the paper that the height of the ordinary truss “could be increased to 33 feet without losing weight in diagonals, but the posts would become considerably heavier.”‡ It is expected to show that such a truss could be made somewhat higher even than 33 feet, with a saving in material. The fear that the increased weight of posts will cause loss when increasing the height, is generally prevalent, and it is not known that any method has heretofore been published, by which the economical height, everything considered, can be obtained at once, without a series of tedious trial operations. Some progress has been made, tentatively, as at least one large manufacturing firm is increasing the height of its bridges, compared with ordinary practice, and recommending still greater heights.

* Treatise on the Theory of the Construction of Bridges and Roofs. De Volson Wood. New York.

† CXXII.—Application of the Theory of continuous Girders to Economy in Bridge Building. C. Bender. Vol. V, page 147. ‡ Vol. V, page 181.

§5. In investigating the subject, a system has been developed in which the volume or weight of any girder is expressed by an equation, of which the minimum value may be obtained by a simple original method, and thereby the proper height of girder and angle of diagonals be determined, to secure the minimum weight or cost of material. In such expression, the lengths, angles, strains and sections of all the members of the girder are so balanced one with another, that each has its effect in securing the result.

§6. The method is applicable to any system of loading, uniform or otherwise, and takes account of the increased sections necessary for material in compression. So also the difference in cost per pound, if any, of compression and tension material, and the comparative costs of different materials which may be substituted in certain locations for iron, wood and stone, for instance, may be taken into account, when the height will be ascertained at which the relative amounts of the several kinds of materials used, will be so proportioned as to secure the minimum cost. The system requires the use of no mathematics higher than algebraic equations of the first degree.

§7. The general result appears to be that the height of all forms of bridge, except the continuous girder, may be materially increased—as compared with ordinary practice—with a saving of material, even when proper provision is made to secure the stability of the longer struts and counteract the increased effects due to wind pressure. In fact, the ordinary heights of some forms of girder may be nearly or quite doubled without loss.

§8. It is not possible, during the time available at the Convention, or in any reasonable space in the Society publications, to explain the method in full, but it is proposed to develop its general principles, and then show the results of its application to various elementary forms of truss and a few of the well known combinations used in long girders.

§9. The investigation was initiated by a graphical method in which the strains were represented by lines at right angles to the direction of the members, a system found convenient in dealing with fluid pressures, as mentioned in a discussion of the subject of bridges at a meeting of the Society, February 16th, 1876, and afterward developed more at length in a paper on Connected-Arc Marine Boilers.* When

*CLXI.—Connected-Arc Marine Boilers, a Demonstration of the Principles of their Construction. C. E. Emery. Page 169.

strains are represented in this way, there is enclosed a figure, the sides of which are severally equal to and at right angles to the sides of the well known force polygon, but the method possesses, in analysis, some important advantages, as the strain diagrams can be laid down without confusing the drawing, and the mind, after a little practice, has a better conception of the magnitude and direction of the forces.* The method is, moreover, capable of direct demonstration on statical principles. If we represent by the several sides of a polygon, the intensity of several forces acting normal to such sides, and such polygon be considered as one base of a right prism subjected to fluid pressure, the prism will be held in equilibrium by the pressure on the several faces, and it is evident, in order to ensure and preserve such equilibrium ;—*first*, that the forces must all act either outwardly or inwardly in respect to the space enclosed, and *second*, that the angles remaining the same, any increase of pressure on one face, must be balanced by a proportional increase on all the other faces. Again, the main prism may be divided into smaller prisms, or joined to one or more other prisms and similar considerations applied to the new sides thus formed ; so, keeping this conception in mind, the system is particularly well adapted for the combination of a large number of strains and a number of force polygons to secure a common resultant. It, moreover, can be readily applied for forming equilibrium polygons and the solution of numerous statical problems, with the same accuracy as if the strains were laid off on lines in the directions they are acting, and with a more ready conception of the several processes, thereby enabling solutions to be obtained with greater certainty and facility.

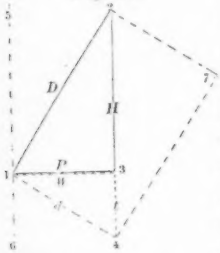
§ 10. The above is evidently only another way of graphically representing the relations of strains, which can be done satisfactorily in the common way. The plan is believed, however, to possess the advantages named, and its use led to the more important development of the expression for the volumes and weights of members previously mentioned. The latter was derived by combining a strain diagram of the kind above referred to, with a diagram of lengths—a drawing representing center lines of bridge members for instance—and ascertaining the relations of the several lines and areas in the combined diagram geomet-

* The writer has recently learned that in wooden bridge building, calculations have been based on the fact that the strains are proportioned to the sides of the angle blocks at the junctions of the members.

rically. In this way, the strains and volumes can be easily obtained graphically by actual measurements, but are usually expressed algebraically.

In Fig. 40, let D , represent a diagonal bridge member with a horizontal projection P , and vertical projection H , then, if we represent a load acting vertically, on either end of the diagonal D , by a horizontal line $W = P$, the strain on D , parallel to its length, will be represented by d , drawn at right angles to D and having the same horizontal projection $W = P$. The shearing strain transferred to or through a vertical member H , will be represented by W , while the thrust or strain received by

Fig. 40.



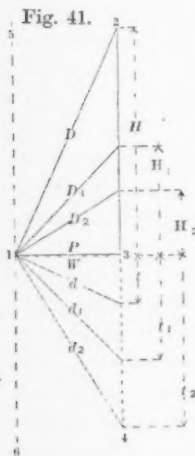
a horizontal member P , will be represented by the horizontal projection t , of the strain diagonal d . We may then call that part of the bridge drawing showing D , H and P , the diagram of lengths, and the triangle dWt , the strain diagram, and by multiplying the length of any member in the diagram of lengths by the length of the corresponding line in the strain diagram, we have an expression proportioned to the volume of the member, from which the volume,

weight or cost may be ascertained by the use of a proper co-efficient. Designating such co-efficient s , we have volume of D or D_v equals $s(Dd)$ equals s times area of rectangle 1, 7. But the triangle 1, 2, 4 (equals half of rectangle 1, 7) is measured not only by half the product of D by d , but also by half the product of W by the line 2, 4 or $H + t$. Hence $D_v = sW(H + t)$. The volume of a vertical member H or H_v , is represented by sWH , and the volume of a horizontal member P by sPt equals sWt . The joint volumes of the vertical H and horizontal P , are then equal to $sWH + sWt = sW(H + t)$ or the same as the volume of the diagonal D .

§ 11. Evidently the volumes of any number of diagonals having the same angle, and of any number of vertical or horizontal members, transferring to or receiving strain from diagonals having the same angle, may be combined by providing the several terms of H and t , representing the volumes of such different members, with co-efficients representing the strains on each in terms of W , and then summing such co-efficients. For instance, if one diagonal, one vertical and one horizontal member be subjected to the strains required to hold in equilibrium a load W , the

expressions for same, viz., $sW(H+t)$, $sW(H)$ and $sW(t)$ may be summed with any number of other members under different strain, say another vertical under double strain, and the expression for the latter being $sW(2H)$, that for the four members jointly will be $sW(4H+2t)$.

It can also be shown that expressions for diagonals of different inclinations and those for horizontal and vertical members under correspondingly different strains may be summed together by simply varying the co-efficients of H and t . Referring to Fig. 40, in the right-angled triangle 1, 2, 3; $P^2 = Ht$; hence $t = \frac{P^2}{H}$ and the value of t varies as the square of P , and inversely, as H ; so, if the angle of the diagonal be changed, thereby changing the relations of P and H , we know also the corresponding change in t which enters into the expression in terms of P and H . For instance, when H is constant, the expression for the volume of a diagonal crossing two panel spaces is $sW(H+4t)$, and for a diagonal crossing half a panel space, the volume equals $W(H+0.25t)$. By equally simple methods, the volumes of diagonals at any angle and of horizontal and vertical members with strains depending upon same, may be expressed in



terms of H and t multiplied by a constant, and as all the members of a skeleton girder must be either vertical, horizontal or inclined, and subject to definite strains depending on the angles of the diagonals and such groupings and concentrations of strain as are due to the transfers of the shearing strains and the moments resulting therefrom, in the particular type of girder,—it follows that the volume of all the members of any form of girder may be expressed in one equation of the general form, $V = sW(\Sigma H + \Sigma t)$ in which the sign of summation Σ , represents sums of the co-efficients of H and t .

§ 12. The minimum value of the general equation may be found without resorting to the calculus, in the following manner: In Fig. 41, if D, D_1, D_2 refer to a diagonal in several positions, d, d_1, d_2 to corresponding strain diagonals, at right angles severally to the first, and

H , H_1 , H_2 and t , t_1 , t_2 corresponding values of the segments of the bases of the several right-angled triangles, we have, with $P=W$ as radius, H =tangent of angle 2, 1, 3 and t =co-tangent of same, and varying the angle, the lengths of main diagonal D and of its corresponding strain diagonal d , become equal when both are inclined 45° , and tangent and co-tangent become equal. At any other angle, either the tangent H or co-tangent t is decreased, but the other, viz., the co-tangent t or tangent H is increased in a greater ratio so that the minimum value of the sum of H and t occurs when $H=t$ and the diagonals are inclined 45° . In the expression for the diagonal $D_r = s W (H + t)$, sW is constant, so the minimum value obtains when $H + t$ is a minimum, or when $H=t$ and the diagonal is inclined 45° .

§ 13. When the volume of a diagonal is a minimum $H=t$, then $nH=nt$, hence, in summing together the expressions of different members, if the terms containing H be kept equal to those containing t , the minimum will not be disturbed and the best angle of diagonals for economy will still be 45 degrees. If, however, the summation result in different co-efficients for H and t , for instance, if $V = sW(mH + nt)$ the expression will not have its minimum value, and if diagonals are inclined 45° , the girder not be of proper height to support, with minimum quantity of material, the strains developed in the type of girder and represented by such co-efficients, but such expression may be made a minimum, keeping the co-efficients constant, by giving new values to H and t , so adjusted that $mH_1 = nt_1$. In such case $H_1 = \frac{n}{m}t_1$. Hence, substituting the value of t previously ascertained, P remaining constant, we have $H_1 = \frac{n}{m} \times \frac{P^2}{H_1}$ or $H_1 = \left(\frac{n}{m}\right)^{\frac{1}{2}} P$. The result is that by making the terms of H equal to terms of t , the most economical height H , of the girder is found in terms of P , which is generally taken to represent the panel length. The final result is a change in the angles of the diagonals and consequently in the strains and sections dependent thereon, in which every member has its influence in producing a minimum and is in length or section or both modified to aid in securing the result in a degree proportioned to its relative value in the original expression.

§ 14. The value of the co-efficient s , may now be deduced. If a bridge member of a certain weight be subjected to a certain strain per unit of sectional area, it is evident that the weight of the member may

be increased, in a certain proportion, either by increasing the length of the member in that proportion, keeping the strain constant, or by increasing the strain in the same proportion, keeping the length constant, as the increase of strain requires a corresponding increase of section and weight. If w equals the weight imposed upon the girder per foot of length, $Pw = W$, the total weight for each panel length, and W is directly proportioned to P . We have heretofore treated P and W as equal, both being represented by the same line. Hereafter, however, we give to each definite values—one in units of length, the other in units of weight—and thereby modify the value of the equation $V = sW(\Sigma H + \Sigma l)$ in direct proportion to the number of units in each. In above equation the portion $W(\Sigma H + \Sigma l)$ represents the product of strains into lengths, and the co-efficient s , may have such value as simply to change the strain factor so that it will represent the section required to withstand the strain, when the expression will equal the product of the lengths and sections, or the volume. It has been found most convenient, however, to use, instead of the section per unit of strain multiplied by the unit of length, the weight of metal required per unit of strain for one unit of length of member. The value of the co-efficient s , which we have termed the *strain unit* to distinguish it from other units, is then as follows :

§15. When the strains are measured in pounds, the lengths in feet and the weight of members in pounds, s_1 ,^{*} for a wrought iron member under a strain of 10 000 pounds per square inch of section, equals the weight of a piece of wrought iron one foot long with a section of one ten-thousandth of an inch, or 0.000 337 44 pounds. With metrical weights and measures, if weights are measured in *tonnes*, the lengths in *meters* and the weights of members in *kilogrammes*, for a wrought iron member, under equivalent strain to that above mentioned, or 7.031 *kilogrammes per square millimeter*, the value of s , s_2 ,^{*} equals 1.1071 *kilogrammes*.

§16. When the strain unit s , is introduced into the expression for a member, it gives the actual weight of the member in denominations corresponding to such unit ; but in the discussion, we shall speak of expressions for the *volumes* instead of the weights, to prevent confusing weights of members with the weights imposed on the girder. When other material than wrought iron is used, or strains greater or less than 10 000 pounds per square inch (7.031 *kilogrammes per square millimeter*) are employed, we have found it most convenient to prefix a numerical co-

efficient to one of the factors in the equation ; for instance, the volume (or strictly, the weight, as before explained) of a wrought iron tension member may equal $D_r = s W(H+t)$ and that of a compression member, $s W(1.25 H + 1.25 t)$.

§ 17. The several values of the strain unit, with logarithms of same, given in the following table will be found convenient for reference as the investigation progresses :

Lengths measured in	Strains measured in	Result in same Denomination as Strain Unit.	
		Strain Unit.	Log. Strain Unit.
Feet (Eng.)	Pounds (Eng.)	s_1^* 0.000 337 44 pounds.	4.528 196 6
Feet (Eng.)	Net tons (2 000 pounds),	s_2 0.674 88 pounds.	1.829 226 6
Feet (Eng.)	Pounds (Eng.)	s_3 0.000 153 06 kilogram's.	4.184 862 9
Meters.	Tonnes.	s_4 2.440 73 pounds.	.387 519 2
Meters.	Tonnes.	s_5^* 1.107 1 kilogrammes.	.044 185 5

§ 18. By the method of procedure herein set forth, it is found that the best angle of diagonals and height of girder to secure minimum weight of material varies with the length of bridge, the number of panels, the horizontal stretch of diagonals, the arrangement of web members and very materially, with the relative strains put upon horizontal and vertical members in compression. As a rule, in long truss bridges, the preponderance of weight in the chords requires for economy that the vertical angles be less than 45° , as will be illustrated hereafter. For all bridges of such size that it is possible to reduce nearly all the sections to a uniform factor of safety under maximum loads, it will be found advantageous to obtain the most economical height and corresponding weight of girders in making the original estimate, which will often be sufficient till construction is to begin, when the several co-efficients of H and t already obtained will be equally valuable in determining the sections required to make out a bill of materials, and the original calculation be thereby, in great part, again utilized.

§ 19. When calculations are made on the basis of a uniform load, the co-efficients of H and t are most conveniently written in terms of one panel load W , and the expression is correct for every length of bridge having a similar number and arrangement of members, except such error, generally small, as arises in volume of bracing due to variation in the

relation of the live and dead loads. When the effects of concentrated loads are to be considered, the maximum shearing strains are used directly as co-efficients of H , and the maximum moments, in terms of the shearing strains, as co-efficients of t , W disappearing. The resulting expression is less general, as a change in length changes correspondingly the system of loading, but is practically correct at various heights.

§ 20. It will be observed that no attempt is made to introduce Gordon's formula or any equivalent of same in the expression, so as to adjust section of compression members to their lengths in terms of their least radii of gyration. Manufacturers generally know by previous trials with formula, what strains are allowable for the particular form of compression member used when of the lengths expected, so approximate calculations can be first made on that basis and afterwards adjusted if necessary. In summing the co-efficients for the different kinds of members, those for the posts are necessarily kept separate, so that the calculation can be revised in a few minutes. For instance, the expression for the volume of half of a truss girder with 14 panels, for one system of loading is as follows: $V = s (3\,127\,900 H + 22\,033\,000 t)$. In this expression, the verticals in compression are represented by $1\,257\,200 H$, on the basis of a strain of 4 500 pounds per square inch; so if, after obtaining the economical height, it be found possible to use a strain of 5 000 pounds per square inch on the posts, the expression for the verticals would be reduced $(5\,000 - 4\,500) \div 5\,000 = 0.1$ or $125\,720 H$, and the original expression becomes $V = s (3\,002\,180 H + 22\,033\,000 t)$. If $P = 18$ feet, the economical height by method shown in § 13, would equal $P \sqrt{\frac{22\,033\,000}{3\,002\,180}} = 48.76$ feet, and the weight of girder for that height per foot of length, independent of the floor, roof, material at junctions, &c., would equal $\frac{S}{7P} (2 \times 3\,002\,180 H) = 784.12$ pounds.* For a height of 36 feet, the corresponding weight per foot would be 878.64 pounds, subject to some corrections which will be discussed hereafter.

§ 21. For arch girders the expression for the volume retains the general form previously set forth, but it has been found convenient to substitute for the term H , a factor of same K , to which are prefixed

* The volume being for half the girder, dividing by $7P$, gives the volume per foot. At the minimum, the terms of H , equal terms of t , so the sum of the two equals two times either. In the above example, the assumed load and consequently the weight of girder is less than is customary for railroad bridges of the length.

co-efficients showing the rate of deflection ; which co-efficients are readily derived from the equation of the particular curve, referred to rectangular co-ordinates.

§22. The general method above set forth is adapted to obtain readily the deflection of any form of girder based upon the actual angles, sections, strains and relations of the several parts. The expression for the deflections is of the same form substantially as that of the volumes, but the co-efficients are determined by a different process.

§23. The length of the paper prevents the presentation of many applications of the system already developed, and of the details of the several processes.

§24. Herewith is presented a table showing some of the results obtained by the application of the system to elementary skeleton girders on the basis set forth in the headings (*q. v.*)

§25. The first example referred to as a "rudimentary arch with earth chord," is a pair of simple diagonals placed like rafters, but to be loaded at the apex. On the basis that the horizontal thrust is balanced without cost, by the reaction of the earth, or that the extra cost of abutments or of a chord is not considered, the proper height for economy is equal to the panel length (column 4), so the proper angle of diagonal is 45° , (column 5).

§26. The numbers in columns 6 and 7, are factors of the actual weights of the girders under conditions named, in terms of sWP . For instance, if the panel length P equals 5 meters (16.405 feet), and the total load W , at apex, equals 66 tonnes (72.752 tons of 2 000 pounds) by using the proper value of s , (§ 16), the weight of each diagonal, on basis of strain adopted (see headings of table), will be found to be 730.7 kilogrammes or 1 611.9 pounds.

§27. When the chord of the rudimentary arch is considered, (No. 2) the economical height of girder rises, thereby relieving the chord until the saving on that account is balanced by the increased weight of diagonals. The economical height, on basis of strains adopted, equals 1.225 times the panel length, giving $39^\circ 14'$ for the vertical angles.

§28. In the upright "king-post" girder (No. 3), with weight applied at bottom of vertical, the section of latter just balances that of the ties, so the vertical angle returns to 45° . This would be the same with suspended king-post, (No. 4,) if the basis of strain in posts and chords were the same. On the basis adopted, however, the economical height is .866 P with vertical angle of $49^\circ 06'$.

ELEMENTARY SKELETON GIRDERS.

Table showing proper heights and angles of diagonals to secure the minimum quantity of material (iron), also the relative quantities of material for the most economical height and for a height equal to the length of one panel. Based on strains, in tension of 10,000 pounds per square inch, (7,931 kilogrammes per square millimeter), and in compression, for posts and diagonals 0.5, and for chords 0.8—the above.

1	2	3	4	5	6	7
Number for Reference.	Sketch of Girder.	Type of Girder.	Girder proportioned to secure minimum Quantity of Material.	Relative Volume and Weight of Material per Panel. (Absolute in terms of sWP .)		
			Height (H) in Terms of the Panel Length (P).	Angle of Diagonals with Verticals.	With most economical Height as per Column 4.	With Height equal to Panel Length P.
1		Elementary Arch—Earth Chord.....	1.000	45° 00'	2.000	2.000
2		Elementary Arch—Iron Chord.....	1.225	39° 14'	2.450	2.500
3		King Post Upright.....	1.000	45° 00'	3.000	3.000
4		King Post Suspension.....	.866	49° 06'	2.598	2.625
5		King Post Upright.....	.817	50° 45'	2.450	2.500
6		King Post Suspension.....	.677	69° 01'	1.732	2.000
7		Triangular Upright, two panels.....	.816	31° 30'	2.449	2.500
8		Triangular Suspended, two panels.....	.816	31° 30'	2.449	2.500

9	10	11	12	13	14	15	16	17	18	19	20	21	22
Quadrangular—Diagonal in tension.....	Quadrangular—Diagonal in compression.....	Triangular—Tie and strut, same inclination.....	Post Triangular—Tie and strut differently inclined.....	Queen Post, Upright.....	Queen Post, Suspension.....	Modified Triangular Upright, three panels.....	Modified Triangular Suspended, three panels.....	Triangular Upright, four panels.....	Fink Triangular Suspended, four panels.....	Fink (old form) Triangular Suspended, four panels.....	Quadrangular, four panels.....	Quadrangular, end posts omitted, four panels.....	Triangular, four panels.....
1.041	1.191	1.000	.990	1.157	1.055	1.528	1.361	1.885	1.700	1.369	.878	1.135	1.013
43 51	40 01	26 34	14 10 37 08	40 50	43 28	44 28 18 07	47 47 20 10	46 42	49 38	55 37 36 09	48 42	41 23	26 16
6.250	7.250	6.000	5.938	5.190	4.690	6.607	5.708	10.250	9.000	8.625	10.187	7.437	6.842
6.246	7.143	6.000	5.937	5.137	4.684	6.112	5.444	8.482	7.650	10.102	9.177	7.377	6.898

Elements required for
one panel of long trusses

§29. When the chord of the king-post is not considered (Nos. 5, 6,) the economical height is materially reduced. This is a practical case when, for instance, the horizontal member is for any reason so heavy that it need not be strengthened to withstand the horizontal component of stress from diagonal, as in floor beams receiving loads direct, and trusses for stiffening compression members. In latter case (No. 6), the economical height is but .58 of the panel length.

§30. The triangular two-panel girders, (No. 7, 8,) it will be seen not only require less height for economy, but also a little less material than the king-post girders on basis of strains adopted.

§31. Referring to the different arrangements of elements required for one panel length of long girders, (Nos. 9-12), the triangular systems require less height and weight than the quadrangular. Of the former, the Post system with struts less inclined than the ties, (No. 12,) is a trifle more economical than when the same are equally inclined, (No. 11).

§32. The differences between the triangular and quadrangular systems would, in practice, not be as great as here shown, as diagonal struts have greater length and are subject, in many cases, to both tension and compression, and therefore require greater proportional sections. The difference varies with the circumstances of the particular case, but may be accurately determined for the same by giving definite values to the quantities, and making corrections in manner previously explained.

§33. In girders with three panels, the "queen-post," (Nos. 13, 14,) is more economical, and has a less economical height than the peculiar form of triangular truss shown, (Nos. 15, 16.)

§34. The comparison for four panel girders, (Nos. 17-22,) is quite interesting. The weight of material in the triangular form proper, (No. 22,) is somewhat less than in the quadrangular form, with inclined ends, (No. 21,) and the latter is more economical than the Fink triangular form, (No. 18,) even at the most economical height of latter, which is nearly half the span. The Fink truss, probably, however, requires the least labor in its construction. The economical height of the original form of the Fink truss, (No. 19), is less than for the other form, so, for a height equal to one panel length, it is a little the more economical of the two.

§35. The economical height of the triangular truss with end-posts, (No. 20,) is considerably less than when such posts are omitted, (No. 21,) the effect of taking the minimum of the equation being to reduce the

length of the posts to the point that the weight saved thereby will be balanced by increased weight elsewhere.

§36. To make the results general, the quantities in columns 6 and 7 have been calculated without considering the permanent weight of the structure. This introduces a slight error on the safe side in the girders (Nos. 13, 14, 20, 21 and 22,) if the quantities in columns 6 and 7 be used as co-efficients of sWP to obtain the panel weights of the girders, W being taken to include both the moving and permanent loads. The excess represents only the extent to which the central and back braces would be relieved of strain by separately considering the permanent load, and is comparatively small for girders with the few panels shown. The co-efficients are, therefore, practically correct for the girders designated, for *any load or length of panel*, on basis of the assumed strains, and are believed to be absolutely correct, on such basis, for the 13 girders (excluding Nos. 9 to 12) referred to in the remaining lines, for which the separate consideration of the permanent load will not reduce the sections of any of the members.*

ERRATA.—On page 227, fourth and fifth lines, for—"it is proper to state that their velocity is so variable, they can seldom be depended upon to move more than the fine rounded sand," read—"it is proper to state that their velocity is too variable to more than say of it, that it has seldom been observed to be less, than that required to move the fine, rounded sand."

*The results of the application of the system to long girders, subject to the influence of extreme concentrated loads, will be presented in a subsequent paper.

THE FLOW OF WATER IN OPEN CHANNELS.

A Paper by THEODORE G. ELLIS, C. E., Member of the Society.

READ AT THE NINTH ANNUAL CONVENTION, APRIL 24TH, 1877.

CALEB G. FORSHEY, in the Chair.

Among the most interesting exhibits in the Swiss section of the Centennial Exhibition, at least to engineers, was the graphic chart of Ganguillet and Kutter for computing the discharge of rivers and canals. An account of the investigations of the above named engineers appeared in the *Cultur Ingenieur* in 1870, and has been translated into French, Dutch, Italian, and more recently, into English, by Jackson. The derivation of their formula, however, is not given in the before named article, but was published in 1869, in the *Zeitschrift des Oesterreichischer Ingenieur und Architekten-vereins*, Austria, in a treatise entitled "Versuch für die Aufstellung einer neuen Formel für die Bewegung des Wassers in Canälen und regelmässigen Fluss-strecken."

These eminent hydraulic engineers have endeavored to deduce from their own and all other available and reliable experiments, a general formula which shall be sufficiently accurate for both large and small streams. The experiments of Humphreys and Abbot upon the Mississippi, Darcy and Bazin, in France, and various other European experiments, differing greatly in size, slope and character of bed, have all been considered. The general result at which they have arrived has been to adopt the well known Chezy formula :

$$V = C \sqrt{K J}$$

and give such a value to the co-efficient C as will adapt it to the different conditions of slope of surface, magnitude of section and character of bed that occur in practice.

By collation and comparison of many selected experiments, they arrived at the conclusion that the velocity varied more nearly with the

square roots of the mean hydraulic radius and the sine of the inclination of the surface, as in the Chezy formula, than with any other powers of those quantities.

They also came in the same manner to the following conclusions :

That the co-efficient C increases :

- 1°.—With the increase of the mean hydraulic radius.
- 2°.—With the decrease of the roughness of the wetted perimeter.
- 3°.—With the decrease of the inclination when R is greater than 1 meter.
- 4°.—With the increase of the inclination when R is less than 1 meter.

Starting with the general formula of Darcy and Bazin,

$$V = \sqrt{\frac{R J}{a + \frac{\beta}{R}}}$$

in which the values of a and β are given for four categories of roughness in the wetted perimeter, they deduce, by empirical methods and comparisons with experiments, the following general conditions, in which : R is the mean hydraulic radius, J is the inclination to unity, V is the mean velocity of the stream, C is the co-efficient in the general formula, and N is the co-efficient of roughness of the bed.

$$V = \left(\frac{Z}{1 + \frac{X}{\sqrt{R}}} \right) \sqrt{R J}$$

$$C = \frac{Z}{1 + \frac{X}{\sqrt{R}}}$$

$$Z = \left(23 + \frac{1}{N} + \frac{.00155}{J} \right) \text{ in metrical measures.}$$

$$X = \left(23 + \frac{.00155}{J} \right) N \text{ in metrical measures.}$$

$$Z = \left(41.6 + \frac{1.811}{N} + \frac{.00281}{J} \right) \text{ in feet.}$$

$$X = \left(41.6 + \frac{.00281}{J} \right) N \text{ in feet.}$$

The co-efficient of roughness, N , varying between 0.008 and 0.050, being the only variable in the formula and remaining the same for all systems of measures.

The values of N , which they give for beds of different materials, are as follows :

Well planed timber,.....	0.009
Pure cement mortar,.....	0.010
Cement, one-third sand,.....	0.011
Unplaned timber,.....	0.012
Ashlar masonry and brickwork,.....	0.013
Rubble masonry,.....	0.017
Canals in very hard gravel,.....	0.020
Rivers and canals in perfect order and free from stones and weeds,.....	0.025
Rivers in moderately good order, having some stones and weeds,.....	0.030
Rivers and canals in bad order, overgrown with vegetation and strewn with stones and detritus,.....	0.035
Irregular beds and broken channels have the value of N nearly equal to,.....	0.050
For the bed of the Chesapeake and Ohio Canal, as described by Humphreys and Abbot, they give,.....	0.033
For the Mississippi River,.....	0.027
For the Bayou Plaquemine,.....	0.029
For the Bayou La Fourche,.....	0.020
For the Ohio River, Point Pleasant,.....	0.021

The chart or diagram referred to at the commencement of this article has been constructed from the foregoing formula, in such a manner, that if any three of the four quantities, C , J , R , N , are given, the other can be found graphically. A similar diagram upon a smaller scale than that shown in the Exhibition, is given by Jackson in his translation.

In order to fully discuss the merits of the foregoing formula and the chart constructed from it, it will be necessary to consider briefly the conditions under which water flows in open channels, and, so far as has been determined by accurate observations, the elements which should enter into the computation of its mean velocity, and the variety of circumstances which affect its rate of flow.

The inclination is generally considered to be one of the chief elements which affect the velocity and rate of discharge. In a long, straight, uniform channel, the accelerating force of the inclination serves only to maintain the velocity of the stream at a uniform rate; the head in a given length being expended in overcoming the resistances of all kinds in that length. If the supply at the head of the channel remain the same, and the inclination be supposed to be increased, the velocity will be increased; but the area of cross-section of the stream will be diminished until the resistances again equal the force of the head per unit of length. If the inclination be supposed to be diminished, the

cross-section enlarges until the resistances are diminished and the same quantity is discharged at a lower velocity. The fact that the resistances generally vary nearly with the square of the velocity, while the force which overcomes them and maintains the velocity, varies as the inclination, causes all streams to acquire an equilibrium of motion where the forces and resistances balance each other.

If now, in such a channel, we suppose that instead of increasing the inclination we diminish the friction in the bed, the same effect is produced; the velocity is accelerated and the cross-section reduced until the resistances again equal the head per unit, to discharge the same quantity; and if we suppose that, instead of decreasing the inclination, we increase the friction of the bed, we shall likewise find the same result: the cross-section of the stream is increased and the velocity diminished to produce the same discharge. We therefore see that the velocity and rate of discharge depend upon the area of the channel and the resistances encountered, correlatively with the inclination.

In natural streams and other large channels, particularly where there are changes in the section of the bed, the inclination does not always remain a function, either of the sum of the resistances or the mean velocity; unaccountable transmitted impulses occur in such streams, and long waves of translation pass along their courses, which materially vary the slope of the surface and leave the true inclination a matter of great doubt. The rise and fall of natural streams also interferes with the maintenance of a slope corresponding to the area, resistances and mean velocity.

In the present state of the science of hydraulics, as applied to the motion of water in open channels, there appears to be great uncertainty as to the manner in which the area and resisting surfaces enter into the formulas for computing the mean velocity. The more common way is to use some function of the mean hydraulic radius, or area divided by the wetted perimeter; while in some cases the surface width, as well as the wetted perimeter, enters into the computation. Which of these methods is the more correct is uncertain, and each has eminent advocates. Until we know the cause of the surface resistance the matter must remain in doubt. If the surface retardation is transmitted from the bottom or sides, as appears probable, the wetted perimeter method may be the best; but if these resistances are independent of the bottom, the surface width should certainly become an element in the computation. Sup-

pose, for instance, we take two channels, each of the form of one half an elongated ellipse, one having the major axis and the other the minor axis for the surface width; one would be wide and shallow and the other narrow and deep; they would each have the same sectional area, the same length of wetted perimeter, and the same mean hydraulic radius, but the surface widths would be totally different; would they, or would they not, discharge the same quantity with the same inclination? If they would do so, the surface width is immaterial; if not, the surface width is an important element.

Of the various theories advanced to account for the surface resistance, none appears to be entirely satisfactory. That which supposes it to be communicated from the sides of the channel appears to be the most plausible, and is supported by the experiments of Darcy and Bazin, from which it appears that, as the sides are brought nearer the centre, the thread of greatest velocity of the current descends. But there are effects, such as the greatest velocity being lower in mid-stream than towards the sides, that this theory does not seem to meet.

All things considered, however, it appears that the form of section should in some way be taken into account in deducing the mean velocity of a stream, from its inclination and the character of its bed.

The roughness of the bed appears, from the investigations of Darcy and Bazin, to be a very important element affecting the mean velocity, at least in streams of the size upon which they experimented. It seems, however, unreasonable to suppose that any of the absolute degrees of roughness that they take into consideration could affect the discharge of a large river like the Mississippi. And one consideration may be assumed with certainty, which is, that for streams of infinite section all the different degrees of roughness given should produce the same result in the formula. This the formula of Ganguillet and Kutter does not do.

Irregularity of section, or difference of area and form in successive sections, appears to be a much more important element for natural streams. The resistances occasioned by eddies and the vertical and lateral currents, as well as boils produced by changes of section, have undoubtedly a great influence upon the mean velocity, and the nature and action of these disturbances is a very difficult matter to determine.

These resistances might be included to a certain degree in the co-efficient of roughness, properly introduced into the formula; but, in this case, instead of having a co-efficient of absolute roughness, it would

appear more rational to make the ratio of roughness, or relative size or depth of the obstructions to the mean hydraulic radius, enter into the formula in the place of the absolute size of the irregularities.. In this manner the great disturbances in large streams might be taken into account, while the absolute size of the projections or roughness of bed, which has been experimented upon, is not of the slightest possible account.

The empirical formula of Ganguillet and Kutter, being deduced from the results of a great number of experiments by different persons, upon streams of all sizes, is now considered the best general formula known to the profession for obtaining the discharge of any stream from its area and section, combined with the extremely uncertain element of its inclination.

They did not, however, go farther in their investigations than merely to modify a previously obtained formula for small streams. It is shown by their own investigations, that making the mean velocity depend upon the \sqrt{RJ} involves a large and uncertain error; that the value of N must be assumed by the "guess and allow" and "average judgment" methods, which, though they may be excellent rules for the experienced engineer, are very poor ones for the novice.

The value of the formula also depends upon the correct measurement of the inclination of the surface, an element which it is very difficult to obtain correctly for large streams of slight inclination. When it is considered that, in measuring the slope of a large river, the ordinary errors of the most careful levelling are a large proportion of the whole fall; that the variation of level in the cross-section of the surface is often as great as the slope for ten miles or more; that the exact point where the level should be taken is often uncertain; that the rise and fall of the water makes it extremely difficult to decide when the levels should be taken at the upper and lower points; that the waves of translation, from accidental causes above and below where the measurement takes place, may affect the inclination to a great and uncertain degree, and may even make the surface slope the reverse way; it will be apparent that in many streams the computation of the velocity from the inclination is a matter of very great uncertainty, to say the least.

The formula of Ganguillet and Kutter is also defective in not taking into account the form of cross-section and irregularities of bed which cause resistance to motion, other than mere roughness of the material of which it is composed. Although the results of experiments upon large

ivers were made use of in preparing their formula, they did not take into account the largest and principal resistance to motion in such streams, but ascribed to roughness of the material of the bed all the resistances due to irregularities of section, of which latter they took no notice.

We find, therefore, that for a river like the Mississippi, having a large area of cross-section and a small inclination, very great differences of velocity and discharge, as computed by the above-named formula with different values of N , when in fact the volume of discharge ought to be almost independent of the roughness of the material of the bed.

Taking the area and inclination at Carrollton, as given by Humphreys and Abbott, where A is 193 968, P is 2 693 and J is 0.00002, we have $R = 72$, $\sqrt{RJ} = .03795$, $\sqrt{R} = 8.49$. Substituting these values in the Ganguillet and Kutter formula, with $N = 0.030$, as given by them for rivers and canals in average order, and we have, $V = 5.60$ feet per second. If we make $N = 0.010$, as given for a bed lined with cement, we have, $V = 11.35$ feet per second, giving more than double the discharge for the difference in smoothness of the bed.

Now, there is no hydraulic engineer of experience who believes that if we should line the bed of the Mississippi with a coat of cement, we should double the discharge with the same area and inclination. The formula, therefore, for large rivers or canals is apparently useless. The true mean velocity for the case of the Mississippi above given, is 5.93 feet per second,* so that if the proper value of N be taken, an approximate value of the mean velocity is obtained. This value of N , however, does not depend upon the smoothness of the bed, except perhaps in a slight degree, but upon other characteristic changes and irregularities in the beds of rivers, independent of mere smoothness, such as the bends and the changes in the form of successive sections.

While the value of $N = 0.030$ is probably nearly correct for large rivers, irrespective of the character of the bed, it would appear to be just as incorrect for a large canal with a straight and regular channel. The value of N would evidently be much smaller in the latter case, even if the bed were rougher.

The irregularities of natural streams being generally proportioned to their size, no great discrepancies were shown in the cases taken by Ganguillet and Kutter in the preparation of their formula; and they as-

* Physics and Hydraulics of the Mississippi, page 316.

sumed that the co-efficient of roughness entered largely into the value of the mean velocity, when the resistances were really due to quite another cause.

The formula of Ganguillet and Kutter, being based upon all the available published experiments of other observers as well as themselves, is very valuable for the smaller streams, and with a proper value for N , is undoubtedly useful for the larger rivers and canals. But this value is too dependent upon the judgment of the observer, and does not altogether relate to the roughness of the surface of the bed.

It is to be regretted that they did not go farther into an investigation of the laws of flowing water, and ascertain the effects of all the resistances affecting the discharge, rather than to take into account only those which affect the smaller streams and canals. Their formula is based upon experiments, and it is consequently far in advance of any of the antecedent empirical formulas, whose inaccuracy has long been known to engineers, but it is so dependent upon the value of N that the true mean velocity due to the given inclination and the character of the bed, will often be a matter of doubt.

The determination of the discharge of a natural stream from its inclination, for reasons before given, is a matter of so much uncertainty that it should never be relied upon where the stream can be accurately gauged, but for ordinary canals and all artificial channels, an inclination formula of as great accuracy as possible is indispensable, and it is believed that for such the formula of Ganguillet and Kutter, notwithstanding its imperfections, is the best that has yet been presented.

A NOVEL RAILROAD SURVEY.

A Paper by THOMAS S. HARDEE, C. E., Member of the Society,

READ AT THE NINTH ANNUAL CONVENTION, APRIL 24TH, 1877.

CALEB G. FORSHEY, in the Chair.

I herewith present for the information and consideration of this Society, the features of what I shall term a "novel railroad survey," which was conceived and executed under my management, over certain portions of the States of Mississippi and Tennessee.

The circumstances attending the necessity for, and the objects to be attained by such a survey were as follows, viz.: the original field notes of the New Orleans, Jackson & Great Northern Railroad, and of the Mississippi Central Railroad, embracing a distance of 441 miles from New Orleans to Jackson, Tennessee, were destroyed during the war, and this destruction obliterated all official data in the possession of these two companies, with regard to the alignment and gradients over that entire distance. It happened, however, that the location of the roads, through the different sections and townships of land, had been preserved by various private parties, and the notes for the entire alignment, therefore, did not prove very difficult to be restored. It also fortunately happened that the gradients, or a copy of the notes showing the undulations of the track from New Orleans to Jackson, Mississippi, a distance of 183 miles, had been preserved in my private possession as a former attaché of the road, so that the only information to be regained, under the circumstances, was the profile of the roadway between Jackson, Mississippi, and Jackson, Tennessee, a distance of 358 miles. Subsequent to the war, and after the consolidation of the two companies, under the present title of the New Orleans, St. Louis & Chicago Rail

Road, it became necessary and important for the economical transit of loaded trains from Cairo to New Orleans, to be possessed of positive information with regard to the intervening gradients or undulations of the track between these terminal points. With the view of obtaining, or restoring, this information, the General Manager* applied to me to devise some means by which to accomplish with expedition, the required work.

It was not desirable to adopt the slow and tedious method of an ordinary survey through the agency of a leveller and one rodman (as would have been necessary, if an original survey, over an unexplored route, had been demanded); but as the principal object to be attained was expedition in the execution, and economy in the cost of the survey, I devoted myself to studying out the following described plan, which was carried through to a successful termination. It resulted in a continuous line of levels being run over the railway route, from Jackson, Mississippi, to Jackson, Tennessee, a distance of 358 miles, in less than eleven days, or an average of over 32½ miles per day, and this, too, during the month

Fig. 42. of July, or during the hottest and most trying season of the year in this climate.



As a part of the preliminary arrangements for embarking on the projected survey, I had a light and suitable hand car prepared at the machine shops of the company, and also had constructed, two levelling rods after a peculiar pattern and design, the principal features of which were that they would offer unusual facilities for noting the reading from the instrument, without the delay in waiting for the tally of the rodman—or they were what may be termed as improved self-reading rods.

The rods were constructed after the following plan, Fig. 42, so as to offer as few figures as possible on their faces to confuse the eyesight. The feet measures were painted in red, and designated in Roman characters, and the centre of the figure was the reading for the even foot. The intermediate figures, representing tenths, were in plain black, and so arranged that the centre, bottom and top of each, represented an even tenth of a foot. For instance, the reading of 1.3 feet on the rod, was the centre of the plain black 3, above the Roman I. 1.4 feet was the

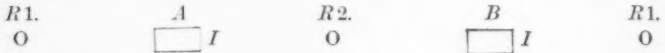
* E. D. Frost, C. E., Member of the Society.

top of the same black 3, and 1.2 the bottom of that figure. In the same way, the bottom of the black 7 was 1.6 feet, the centre, 1.7 feet, the top, 1.8 feet, and so on.*

In the organization of the surveying party, therefore, I had the following force and outfit; one hand car, two levelling rods, and four assistants, two of whom acted alternately as rodmen and propellers of the hand car.

The initial point at Jackson, Mississippi, was taken from an established bench mark, immediately on the line of the road within the depot grounds, and this bench mark showed a reading of a certain number of feet above tide level as projected originally from the seaboard.

The situation at the start can be exemplified by the following diagram, viz.:



In this connection, I would state that, as the alignment, including tangents and curves, was not in question, there was no pressing necessity for a re-survey in this direction, and as the mile posts were established over the whole route, as checking points, at stated intervals, the counting of the bars of iron, which were of uniform lengths, in the selection of turning points, enabled the survey to be conducted with all the practical precision required for a profile of the gradients as they bore a relation one to the other.

At the start, the forces were thus distributed: *R1* represents the position of rodman No. 1; *A* represents the hand car at a distance of 400 feet north of rodman No. 1; *I* represents the instrument, planted in the centre of the track immediately in front of the hand car; and *R2* represents rodman No. 2—400 feet north of hand car. The distance between the two rodmen was about 800 feet, whenever the topography of the country would admit of a maximum sight of 400 feet each side of the instrument.

As soon, therefore, as the instrument was adjusted and an observation taken on the rod at *R1*, a signal was given which started rodman No. 1, in a double quick, towards the hand car. By the time the instrument had been reversed and the sight obtained on the rod at *R2*, the first rodman was seated on the hind end of the car, and with the instru-

* The intermediate hundredths were graduated on the rod so as to be easily discernible at a maximum distance of 400 feet. The centre between any 2 feet was designated by a plain heavy black line running entirely across the face of the rod—as for instance, 1.5 feet.

ment folded upon my lap, on a comfortable seat on the front of the car, the two assistants propelled the whole party (with the exception of rodman No. 2, who remained fast) at a rapid rate over a distance of 800 feet to the second position at *B*. As soon as the hand car stopped, rodman No. 1 made a double quick 400 feet in advance to his second position, and by the time the second back sight had been recorded at *R2*, he was in position with his rod for a foresight. This double quicking of the rodmen was kept up until they were fatigued, and their places were then supplied by the two assistants, who were acting previously as propellers of the hand car, and in this way the labor was so evenly distributed that the day's work of 30 odd miles was wound up without any extraordinary fatigue to any one of the assistants. It will be observed that the leveller had no walking whatever to do, and I managed to keep up all notes and calculations as the hand car was passing from one position to another.

The most difficult and trying portion of the survey was the anxiety constantly engendered for the safe passing of all trains approaching both in front and in rear, so as to avoid the disasters of a collision; and to provide against any contingency of this nature, I was furnished at each station with telegraphic information of the probable position of all approaching trains. By this precaution, and by exercising the utmost vigilance, I escaped with no actual accident; but on one occasion it was a mere scratch that a serious one was obviated. It was late in the evening and on a portion of the road where there was a long curve, and where there had been recently laid some new fish bar iron; although I knew there was an approaching passenger train, behind time, I was closing the day's work and ventured too far, relying somewhat upon the ear or hearing, to guide me in the approach of the train. It approached so noiselessly over the new laid iron that I was caught almost in a trap, with a train running at the rate of 30 miles an hour bearing down upon us; but owing to the lightness of the hand car, and the energy and coolness displayed by my assistants, we made a most fortunate and hairbreadth escape. I mention this circumstance only to show the difficulties which constantly surround such a survey, and to show, also, that it was no small undertaking, even in a physical point of view, to take the observations for, and to record the level notes over a distance of 33 miles in a single day.

It may be well to remark that there was no extreme accuracy required in the line of levels to be run, such as would have been necessary on a

canal or water route, and, therefore, the results, although only relative and approximate, answered all that was needed for a practical solution of the questions and interests involved.

MR. WILLIAM H. SEARLES.—By way of analysis of this extraordinary survey, I have been making a few figures which may be interesting to the Society.

Taking the average day's work at 34 miles, each rodman's duty was to run $8\frac{1}{2}$ miles at a double quick, ride $8\frac{1}{2}$ miles at ease, and propel the hand car (assisted by another rodman) for 17 miles per day. I can only express my admiration for the physical endurance which could sustain such a strain with impunity for days in succession, under the scorching rays of a July sun in a southern climate.

Again, with a setting once in every 800 feet, there would be 224 settings in 34 miles, which, at three minutes each, would consume eleven hours and twenty minutes, to which must be added the time necessary for refreshment, and to avoid passing trains.

To mount a car, ride 800 feet, dismount, set up an instrument, take readings in opposite directions and record them, all in three minutes, and to repeat the operation continuously for eleven or twelve hours a day, is almost as remarkable a feat as that of the rodmen.

I offer these remarks just as they have occurred to my mind—not at all in a censorial spirit, but simply for the purpose of lending additional interest to this latest achievement in modern engineering.

MR. THOMAS S. HARDEE.—In reply, I would state for the information of the members of the Society, that my assistants were four able-bodied *Africans*, who were fully capable of enduring the amount of fatigue and physical labor so graphically pictured by Mr. Searles, and who were also sufficiently intelligent to acquire almost immediately, all the mechanical skill needed in the handling of the rod.

I doubt if the same amount of work could have been performed in the same time and under the same circumstances with white labor, and therefore, I consider that the success of my enterprise, as a novel feat of field engineering, depended in a great measure upon the use of African muscle and its peculiar application in the manner described.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CXLVII.

PROPORTIONS OF EYE-BAR HEADS AND PINS,

AS DETERMINED BY EXPERIMENT.*

A Paper by C. SHALER SMITH. C.E., Member of the Society.

PRESENTED JUNE 20TH, 1877.

The best method of commenting upon Mr. Burr's very carefully prepared paper on stresses in the eye-bar head, is to present the results of a series of trials made with eye-bars of the usual sizes in standard structures, for the purpose of determining practically what Mr. Burr has endeavored to do mathematically. In experimenting on this subject during the years extending from 1857 to 1866, I had ascertained, as an absolute unchangeable fact, that a pin, the diameter of which was 66 per cent. of the width of the bar, was the least size that would invariably develop the full strength of the bar in testing to final rupture (presuming that the shearing stress has been properly provided for). Also, that the required metal section across the eye was a variable quantity, and depended on the relative proportions of the pin diameter and width of bar. Also, that the shape of the eye-bar head required to give a

* In discussion of—CLX; Approximate Determination of Stresses in the Eye-Bar Head; W. H. Burr. Page 127.

slightly higher factor than the bar itself, depended entirely upon the material and mode of manufacture.

In 1868, I broke 57 bars with pins and eyes of varying sizes, in order to determine accurately the laws governing the breaking strengths of hammer-forged eyes subsequently welded to the bars. The results of these experiments are given in the table (page 266) under "No. 1, Hammered Eyes." These trials developed the fact that for hammered eyes there are two points to be fixed, the section behind the pin and the section across the pin, and that when these are determined, the curve passing through these two points and having its centre on the vertical line across the pin, is the proper periphery of the eye-bar head.

In 1875, there were broken at the Edgemoor Works, in the Kentucky river bridge trials, 54 hydraulic forged eye-bars, or weldless links, as they are commonly termed. From these (referring to the table) results under "No. 2, Welded Eyes" have been deduced, as also the further fact that in the hydraulic forged eye there is but one section to fix, to wit, the section across the pin, and that with this once established, the curve of the eye is a simple circle struck from the centre of the pin. In both sets of experiments, the rule was adopted of considering no proportion established until three similar eye-bars had been broken of each size without a failure in the eye.

To recapitulate, the following rules are unvarying :

First.—As the relative proportion of the diameter of pin to width of bar increases, more metal is required in the section *across* the eye.

Second.—In hydraulic forged eyes, this is the weak point and governs the rest of the eye, which is consequently a true circle.

Third.—In hammered eyes, two points must be fixed, the section back of the eye and the section across the eye.

Fourth.—A pin, 66 per cent. the width of the bar, is the smallest which will invariably break the bar, or develop its full strength.

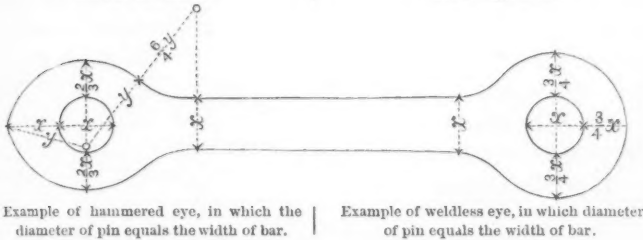
The importance of the first of these facts is apparent at a glance. A middle chord pin of a large span may have attached to it; first, the chord bars to which its diameter may be as 0.75 to 1; next, the tie bars, say as 1.25 to 1, and finally the counter ties, at as 2 to 1. If these eyes are all proportioned as required for the chord bars, the ties and counters will be much too weak; and this is the condition of these members in many large bridges now in existence, owing to the erroneous belief that the metal section across the eye should bear a constant pro-

portion to the section of the bar, irrespective of the method of manufacture and the diameter of the pin.

Fig. 43.

From St. Charles bridge experiments,
made at St. Charles.

From Kentucky river bridge experiments,
made at Edgemoor.



In the above examples, Fig. 43, the thickness of bar and eye are presumed to the same. If they are *not* the same, the factor x , which represents linear distance, as referred to width of bar and diameter of pin, will express *sectional area* when used in proportioning the metal sections across the eye and back of the pin. It should be noted in this connection, that it is not necessary to vary the metal section back of the pin in the hammered eye, as the section " x ," covers all cases in common practice. In the hydraulic forged eye, this section is changed with the thickness across the eye, but this is principally on account of the greater facility in shaping the die afforded by the circular shape, as owing to the condensing of the iron by pressure, there is a surplus of strength at that point.

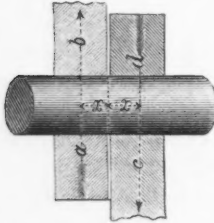
It is apparent from the 111 experiments referred to, that so far as the width of the eye-bar is concerned, the size of the pin is a matter of no importance, provided the metal is properly proportioned in the eye and the pin is not less than two-thirds the width of the bar, but the maximum thickness of the bar is entirely dependent on the relative diameter of the pin. The considerations governing this maximum thickness are as follows :

First, is the fact that the elastic limit of solid wrought iron cylinders in flexure, is greater than the same limit in direct tension and compression. This is true also of cast iron and steel, but we are dealing just now with wrought iron pins. Thus, a grade of iron having an elastic limit of 25 000 pounds in tension, will rarely take a permanent set at less than 40 000 pounds per square inch fibre strain when tested by bending. It is evident, therefore, that where 10 000 pounds per square inch is

the limit in tension, 15 000 pounds per square inch may be used with equal safety as a bending strain.

Next, comes the consideration, that the pin is compressed by and compresses its bearing or its counter eye-bar, just as much as it affects or

Fig. 44.



is affected by the eye-bar selected for calculation, and consequently in determining the fibre strain, the leverage distance should be from centre of eye-bar to a point within the shearline equal to *half the thickness of the eye-bar*. Thus where two eye-bars are on the same pin, as in Fig. 44, the forces should be considered as acting with a leverage of twice the distance x , and the eye-bar stresses should be

considered as concentrated in the lines ab and cd . This principle is well stated by Mr. Bender,* but Mr. Macdonald, in determining the same strain, uses x instead of $2x$.† Fixing 15 000 pounds per square inch as the extreme fibre strain, the following table will give the extreme thickness of eye-bar which can be used on any pin in single shear, the bar strain being 10 000 pounds, and the flexure strain its equivalent, or 15 000 pounds per square inch.

The following table presents, in a compact form, the metal section of the eye-bar head and the maximum thickness of eye-bar for a pin of any size, all dimensions being expressed with the eye-bar width as unit.

TABLE.

Width of Bar.	Diameter of Pin.	No. 1. Hammered Eyes.		No. 2. Welded Eyes.	
		Metal Section across the Eye.	Maximum Thickness of Bar.	Metal Section across the Eye.	Maximum Thickness of Bar.
1.00	0.67	1.33	0.21	1.50	0.21
1.00	0.75	1.33	0.25	1.50	0.25
1.00	1.00	1.50	0.38	1.50	0.38
1.00	1.25	1.50	0.54	1.60	0.54
1.00	1.33	1.70	0.59
1.00	1.50	1.67	0.70	1.85	0.70
1.40	1.75	1.67	0.88	2.00	0.88
1.00	2.00	1.75	1.08	2.25	1.08

* Proportions of Pins used in Bridges; C. Bender. New York.

† LXXIII. Proportions of the Heads of Eye-Bars; C. Macdonald. Vol. I, page 333.

The use of this table may be noted from the following examples:

If a bar 4×1 inches is attached to a 3-inch pin, the section across the eye (table, under "No. 2, Welded Eyes") should be $4 \times 1.50 = 6$ inches, and the maximum thickness of bar, $4 \times 0.25 = 1$ inch. Should the same bar be attached to a 7 inch pin, however, the metal section across eye must be $4 \times 2 = 8$ inches, and the maximum thickness, $4 \times 0.88 = 3.52$ inches. The irregularity in the ratios between the pin-bar widths and eye-bar sections in the table, is due to the fact that the test bars declined to break by formula, and, consequently, only those proportions were used which gave no failures whatever in a sound eye-bar head.

The St. Charles experiments were made on bars varying from 4×1 inches to $2\frac{1}{2} \times \frac{3}{4}$ inches, while those for the Kentucky river bridge were confined entirely to bars, 3 inches in width and of a uniform length.

These last tests are printed in the Report of the Cincinnati Southern Railroad for 1875; the St. Charles tests are still in manuscript.*

So far as I know, the table deduced from these tests is the only one in which the real conditions governing the proportions of eye-bar and pin connections are recognised; these conditions being—*first*, that the shape and proportions of the eye-bar head must change with a varying ratio of pin diameter; *next*, that the manner of making this change depends upon the mode of manufacture; and, *lastly*, that the limit of elasticity in flexure being greater than in direct stress, the thickness of the bar, which is a third proportional to the width of bar and diameter of pin, may be determined with a fibre or bending strain of 15 000 pounds per square inch on the pin.

* A copy will be furnished, if desired.

CXLVIII

WING DAMS IN THE MISSISSIPPI

ABOVE THE FALLS OF ST. ANTHONY.

A Paper by EDWARD P. NORTH, C. E., Member of the Society.

READ MAY 11TH, 1877.

The Mississippi River would be navigable from the Falls of St. Anthony to Pokegema Falls, 360 miles, if it were not for certain obstructions.

In the summer of 1874, \$25 000 were available for the improvement of this portion of the river. Col. F. U. Farquhar, Corps of U. S. Engineers, who was charged with the expenditure of the money, decided to use it in improving the 77 miles between Minneapolis, at the Falls of St. Anthony, and St. Cloud.

The writer was in immediate charge of the work ; all the plans, however, were either originated by Col. Farquhar, or submitted to him for his approval.

The Mississippi between the points mentioned, usually runs between well defined banks that are subject to erosion in but few places, the bed of the stream being generally hard pan or gravel, and there is but

one place where a cut off is possible ; the average fall of the river is 1.3 feet per mile ; there are however several boulder bars acting as dams, giving pools above and rapids below their crests.

The most rapid water during the low stages of the river is at Spring Rapids, full 1.5 feet in 800, or 9.9 per mile; this is surmounted by a "stern wheeler" towing a barge, by taking a run at it.

From St. Cloud to near Fort Ripley, 53 miles, the river passes through an outcrop of crystalline rocks, and will require 3 or 4 locks to render it navigable ; from Fort Ripley to Grand Rapids, $3\frac{1}{2}$ miles below Pokegama Falls, the river flows through an alluvial country, with flat slopes, "cut offs," &c., and gives traffic to one steamboat that plies between Aitkin on the Northern Pacific R. R. and Grand Rapids.

Between St. Cloud and Minneapolis very little sand is carried by the river at low water, and the water is brown from the swamps and wild rice fields at its head.

Just below St. Cloud are the so called Thousand Islands, where the river is spread by a gravel bar into many channels, preventing the passage of boats at low water.

It was determined to improve this by brush dams.

The appropriation, which was not available until the middle of July, was too small in amount, and its continuance was too precarious, to justify any large expenditure for outfit—stone for the dams could not be procured for less than \$2.50 per cubic yard, and all the gravel that could be procured at the outset was in the bed of the river ; for these reasons there were some peculiarities in the location and construction of the dams.

CONSTRUCTION OF DAMS, MATERIAL, ORGANIZATION, OUTFIT, &c.—The dams were built of fascines, secured by stakes, and gravel or sand. 2 735 linear feet of dams and 314 feet of bank protection were built at the point indicated, with contents as follows :

Dams.....	6 807	cubic yards.
Bank protection.....	418	" "
Total.....	7 225	" "

4 699 fascines were used, varying in diameter from 9 to 18 inches, and in length from 15 to 22 feet.

The average height of the dams was 3.3 feet, and 1.62 fascines were used per linear foot.

Allowing the average size of the fascines to be 16 cubic feet each, 38.5 per cent., or 2 785 cubic yards of the dams were brush and 4 440 cubic yards were gravel, which was transported an average distance of over 500 feet, mostly in wheelbarrows.

It was impossible, on account of absence at other works, to take measurements as often as necessary to arrive at correct quantities, and some of those given, depending on the writer's judgment, are liable to a \pm correction.

The work was in charge of H. C. Henry, overseer, who was possessed of industry and judgment; the laborers were divided into those who worked on land at 19½ cents per hour, and those who worked in water at 26½ cents per hour.

The outfit, implements, boats, lumber for horses and runways, spun-yarn, rope, &c., cost \$763.44, and the value of material at the close of the work was estimated at \$137.67, leaving \$625.77 for use, which is apportioned as follows:

Making fascines, including 2 500 pounds spun and lath yarn.....	\$379 31
" stakes	5 40
Laying fascines.....	68 00
Driving stakes.....	50 35
Graveling dams.....	122 71
	<hr/>
	\$625 77

The fascines were made from willow, swamp maple, black and burr oak brush, nearly all but the first being the tops and trimmings of stake timber.

Five men constituted a gang of fascine makers, 1 cutting brush, 1 cutting and carrying, and 3 at the trestles, which were of 2×4 inch pieces driven into the ground and connected at the crossing—which was about the height of one's hips—by a carriage bolt.

The levers for the "chokers" were 46 inches long, 3 inches in diameter at the ring for the chain, which was 18 inches from the short end; they were 1½ inches in diameter at the short end and 1½ inches at the long, and the chain was 6 feet long, which is the right length for an 18 inch fascine. When smaller ones were made, knots were tied in the chain.

The fascines were usually tied with 5 bands of spun or lath yarn, each band consisting of 2 strands wrapt twice around the bundle and tied by the third man in a square knot; while the others bore their

weight on the levers, the bundles were very compact. 4 704 fascines were made, and 5 276 hours' labor were charged to the same, which included the carriage of the fascines to the bank, or some places accessible to teams by the men at the trestles, giving about 65 minutes' labor per fascine. The best work that was noted was at the rate of 1 fascine in 49 minutes.

Brush for about 240 fascines, 15 inches in diameter, can be cut from an acre of willow as it grows on the "tow heads" and islands. I think not more than one-half of this would be cut from burr or black oak brush.

The cost of 4 704 fascines was as follows :

5 274 hours' labor, at 19½ cents.....	\$1 028 82
Use of tools and spun yarn.....	379 31
Superintendence and overseer.....	387 50

	\$1 795 63

or 38.2 cents each; of this 7.5 cents was for binding material, 0.556 pounds having been used per fascine. If the work had been continued it was proposed to use galvanized wire for at least one dam.

The cost of making 14 280 stakes, 5 to 9 feet long, was—

1 885 hours' labor, at 19½ cents.....	\$367 58
Use of tools.....	5 40
Superintendence and overseer.....	109 50

	\$482 48

or 3.4 cents per stake.

PLACING AND STAKING FASCINES.—The fascines were at first tied into mats between parallel poles and rafted into position, but with the appliances we had, it was difficult to place them satisfactorily, in consequence of the rapid current, and after 2 or 3 mats had been sunk, the fascines were transported to the dams on scows and placed separately.

One row was generally placed across the bottom, parallel to the current, brush ends up stream, and staked back of the upper band. A single row of fascines was then placed lengthwise of the dam, about 3 feet up stream from the butts and staked through each bundle under it. This was advisable, because the current bent back the tips of the fascines so that the dam tended to become thickest in front. Gravel was then filled in to cover the tips of the lower row and make a plane surface to the transverse fascines. The succeeding courses were placed parallel to the current, except where the dam was unusually high, when

another transverse course was placed, care being taken to have at least one course parallel to the current when the dam was finished. When the current swept the gravel off the dam as it was being built, boards about a foot wide were slid down in front of the back row of stakes, which retained the gravel until the fascines were secured on it, the intention being to have about a foot of gravel between the layers of fascines.

The courses, as usual, were stepped forward $2\frac{1}{2}$ to 3 feet. In one instance, on a dry bar, a single row of fascines were laid lengthwise of the dam and a layer staked across them, with the butts overhanging about 2 feet. This plan did not prove judicious, and was abandoned.

Stakes were driven, if possible, into the bed of the stream and to the surface of the water or dam.

On the completion of the fascine work, any projecting stakes were cut off nearly flush with the surface of the dam, so that they should not be loosened by logs striking them. All was then covered with a coating of gravel carried horizontally about 6 feet up the stream. It was intended to give the fascines such an inclination that the depth of gravel would be fully a foot at this point, to protect the dam from the impact of logs (of which 225 000 000 feet B. M. were floated down the river during 1874), also to guard against the fascines being torn out, in high water, by the buoyant effect of the ice frozen to their butts.

The cost of placing and staking 4 699 fascines was :

4 290 hours' labor, at $26\frac{1}{2}$ cents.....	\$1 136 85
507 hours' team work hauling fascines, at 40 cents.....	202 80
Use of tools, laying fascines.....	68 00
“ “ driving stakes ..	50 35
Superintendence and overseer.....	529 36
Total.....	\$1 987 36

About 65 per cent. of the above labor and superintendence was probably expended in driving stakes.

GRAVEL.—Very little of this would have been stopped by a $2\frac{1}{2}$ inch ring; 40 or 45 per cent. was sand and the rest gravel, in the usual acceptance of the term.

When operations were commenced, gravel could not be obtained from the banks, and dam No. 1 was so located as to leave as much of the bar below it available for its construction as possible. Dam No. 2 was also given its curved outline, partly for the gravel and partly for economy in construction.

Before No. 2 was completed, permission was obtained to take gravel from the bank opposite the head of Island 126, which was taken over the channel on runs supported by horses.

Dams 3 and 4 were built almost entirely of fine river sand, covered and faced with gravel brought on scows. Dams 5 and 6 were built with gravel laid dry by themselves and Dam No. 1, which was brought on scows and run-ways.

COST OF 4 440 CUBIC YARDS OF GRAVEL :

Loading, wheeling, &c., with care of runs on ground, 11 727 hours, at 12½ cents.....	\$2 286 76
Setting and maintaining runways over water, 428 hours, at 26½ cents...	113 42
Use of tools, &c.....	122 71
Superintendence and overseer.....	743 22
Total.....	\$3 266 11

or 75 $\frac{8}{10}$ cents per cubic yard.

TOTAL COST.—As stated above, there were 2 735 feet of dam and 314 of bank protection. As 1.88 feet of bank protection was equal to 1 foot of dam in contents, the length of dam may be called 2 900 lineal feet, and the quantity 7 225 cubic yards. The cost was found to be as follows :

Making fascines.....	\$1 795 63
“ stakes.....	482 48
Placing fascines.....	695 59
Staking “.....	1 291 77
Gravel.....	3 266 11
Total.....	\$7 531 58

Cost per lineal foot, \$2.60 ; cost per cubic yard, \$1.05.

About 34 per cent. of the cost was for gravel, and 24 per cent., or \$1 775, for stakes and driving them.

This would have bought 710 cubic yards of rock ; about twice that amount would have been required to anchor the brush, and it is doubtful if the dams would be as tight as at present.

Stone laid on the faces of the dams would be picked up by the ice and carried down stream. A more expensive outfit would also have been required to control the mats in the current until they were anchored. On the other hand, the quantity of gravel used would have been reduced.

The only dams with which I am sufficiently familiar to institute a comparison, are those built under the direction of Col. Houston by John

Nader and his successors, which are proving so efficient in controlling and deepening the channel of the Wisconsin.

In the report of the Chief of U. S. Engineers for 1872 (page 132 *et seq.*), Mr. Nader returns the cost of 10 679 feet of dam, "not including stationery, mileage, tools, boats, or general repairs of tools, boats, or implements," as \$41 042.06, or \$3.81 per lineal foot. Of this, \$11 590.36 is charged as materials, probably brush and stone, which would reduce the cost of building to \$2.76 per foot. No estimate is made of the cost per cubic yard.

The system adopted on the Wisconsin, is the use of fascines 10 inches in diameter and 12 feet long, tied by 3 wythes, which are made into mats, sunk and anchored by the use of stone.

The estimated cost for 1876 is \$3.50 per foot, but no statement of cost has been made public since 1872.

The price of the Mississippi river dams, \$2.60, not only included all of the above items (excepting stationery), excluded by Mr. Nader, but also contained the cost of a preliminary survey. It is possible that the use of stakes instead of rock may often be economical, notwithstanding the rather heavy cost of driving them.

The dams at St. Cloud were intended to increase the depth of water on bars near Islands 126 and 129, and to protect a sand bluff behind 137, which contributed a great deal of sand to the river in high water.

On the completion of the last dam there were about 3 feet of water on the bar at the head of Island 126, and an average gain of over 1 foot below that, which improvement was thought due to both erosion and the increased amount of water in the channel, there being a decided back-water from Dam No. 2 to above the lower landing. The accompanying map (Plate XVIII.) is from surveys made before operations had altered the depths of the water.

The dams were left about a foot above low water, as it was not thought that the river carried sand enough to fill the channel during high water, and it was desirable to present as little obstruction to log driving as possible.

No attempt was made to use the whole volume of the river, as the water naturally flowing by the lower landing was thought sufficient to make a 3 or 3½ foot channel; the amount of water, however, can be readily increased by dams near Island 109.

The locality was visited in the latter part of December, after the river had frozen. Dams Nos. 1 and 2 were in many places covered with

thick ice, which extended over the rear of the dams, enveloping the butts of the fascines ; the dams were unexpectedly tight, only one leak in the first and three small ones in the second were observed, and there had been a decided shoaling in front of the concave part of No. 2 ; the other dams were so thoroughly covered with ice that not much could be seen of them.

The winter of 1874 and '5 was a severe one ; at St. Paul the thermometer ranged for about twenty mornings between 0° and -39° Fahr ; ice in the river was 3 feet and in some instances 4 feet thick. Mr. Henry was at the dams shortly after the break-up, and learned that the ice first followed the channel to dam No. 1, on which it gorged ; the ice then broke over Nos. 2 and 5, passing behind Island 137, cutting a good deal of sand from the bluff. None of the dams showed any injury, except that the butt of the upper fascine in Dam No. 4, which was mainly built with sand, had been lifted and bent over by the ice ; this was very satisfactory, as no crib dam had stood on the river from the Falls of St. Anthony up.

If the improvement of the river is ever undertaken, with a prospect of continuous appropriations, it will probably be best to place a sufficient number of light dams in the channel back of Island 137, to warp it full, and then plant willows on the bar so formed, as the bluff is the worst reservoir of sand on that portion of the river.

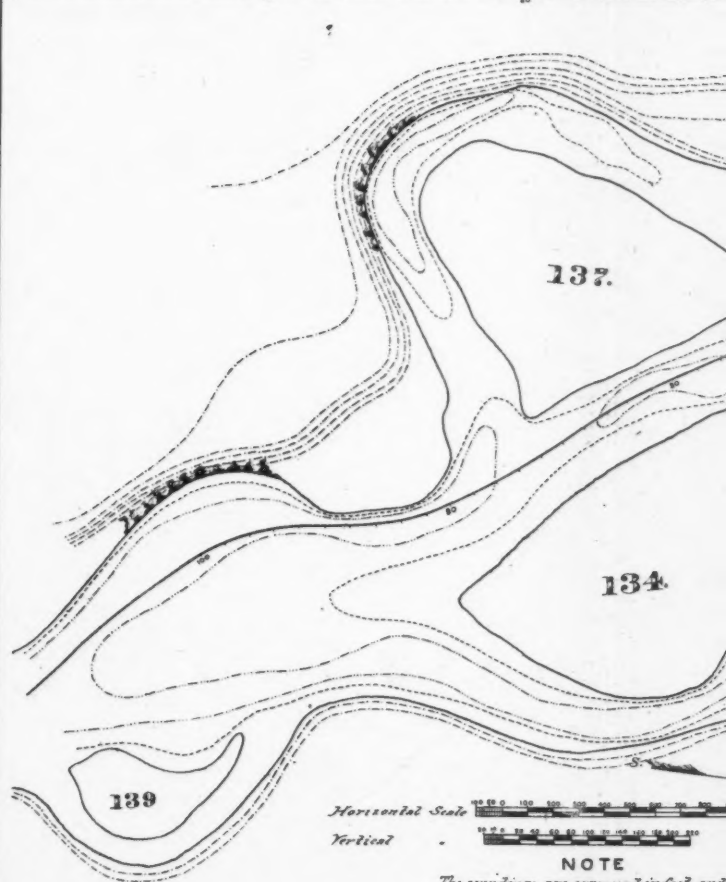
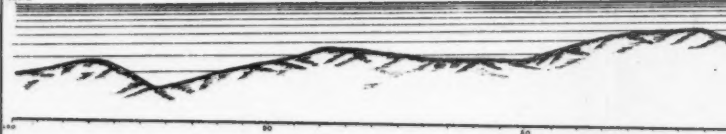
An attempt was made to close the cut-off below Clear-Water. The water here was from 10 to 12 feet deep and the current very strong. Stakes or light piles were driven by hand from a boat, in pairs about 10 feet apart ; horizontal pieces were spiked to the stakes, and poles lashed to them, on which fascines were placed and loaded with gravel (the horizontal pieces carrying the run-planks for the wheelbarrows) ; the lashings were cut simultaneously and the mat allowed to sink ; this was repeated till the dam was brought to the surface of the water. The increasing cold and a rise in the river flooding our gravel pits, compelled the abandonment of the work before it was secured. It was intended to sink an apron of mats at the rear of the dam, which was nearly vertical, and shingle mats down the face. The next year, the river men cut through the bend at the end of the dam, to save driving their logs around the bank, which was sufficient to wreck the affair completely, and the main channel of the river now follows the cut-off.

Since writing the above, Col. Farquhar writes to me, that on April 15th, an inspection of the dams was made by J. L. Gillespie, Assistant Engineer, who reports in substance as follows :

Dam No. 1 has three breaches in it of an aggregate length of 73 feet. Dam No. 2 has a breach about 30 feet wide opposite the upper point of island No. 126 ; the rest of the dam is uninjured and tight ; that portion between Islands 112 and 123 being well covered with sand and small willow brush growing upon it. The upper layer of fascines for nearly the whole length of Dam No. 4 seems to have been torn off, leaving the stakes about a foot higher than the dam. There is also a breach 35 feet wide at the head of Island 137. Dams Nos. 3 and 5 are in good condition, with a sand bar 10 to 40 feet wide behind No. 5. Mr. Gillespie adds "as there has been no high water or heavy ice since the last inspection, it is probable that all the breaches above described were made by log drivers during the past summer and fall." Col. Farquhar remarks "they have stood splendidly and would be as good to-day as when they were put in, had it not been for the vandalism of the log drivers who made the cuts through them."

It will be remembered that Dams Nos. 3 and 4 were built of fine sand, covered and faced with gravel ; the gravel in Dam No. 1 was as coarse, if not coarser, than in any other.

457.0




Horizontal Scale





Vertical

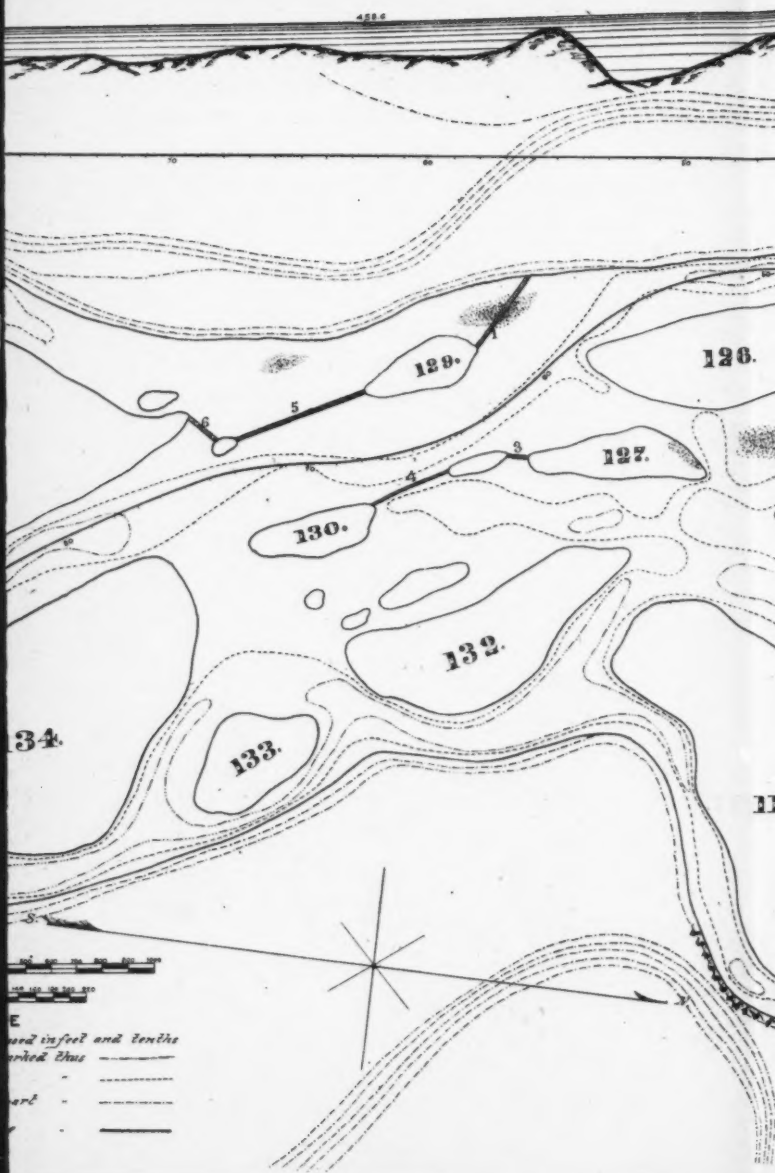


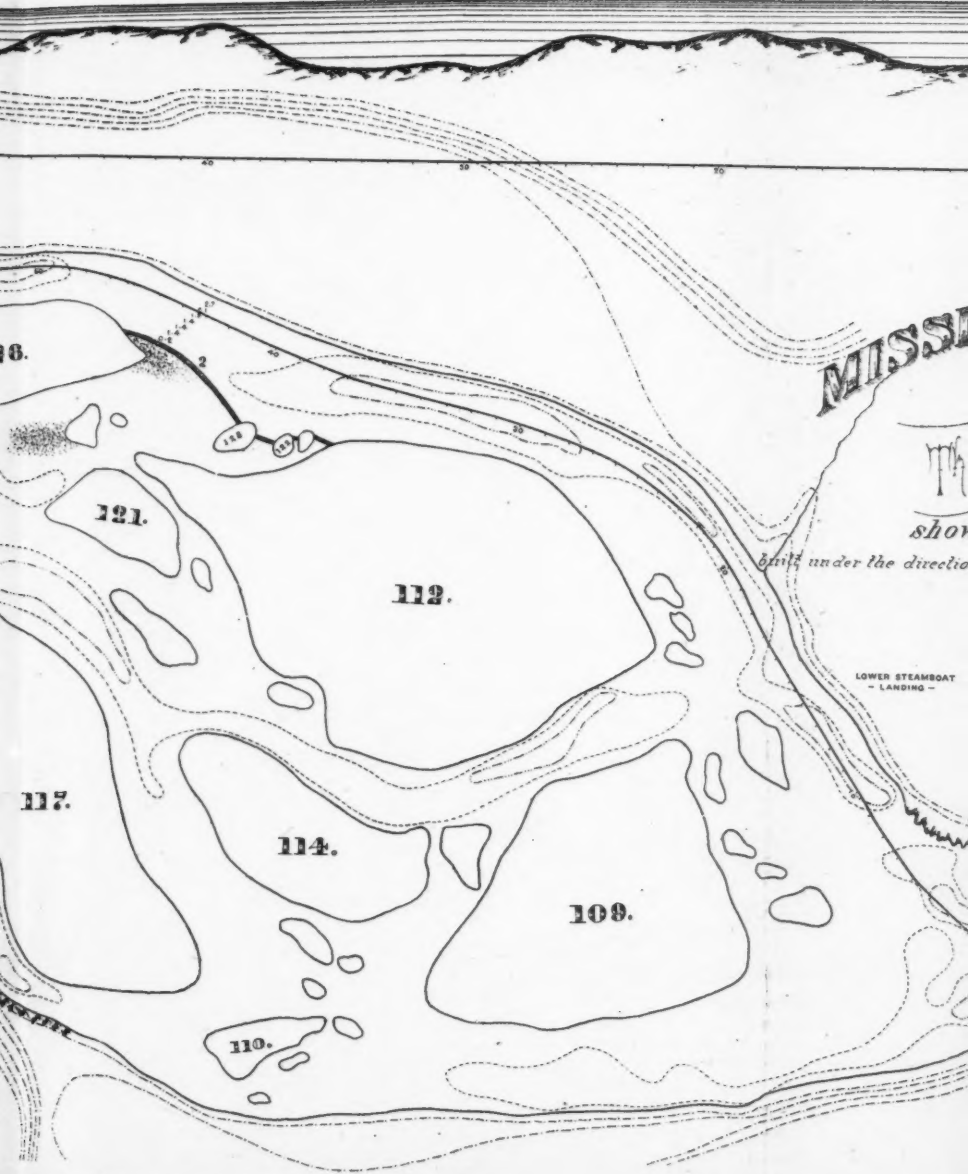
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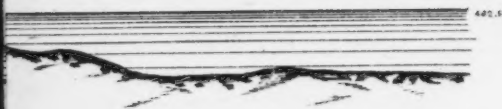
The soundings are expressed in feet and
Water 5 feet in depth marked thus 

Contour lines 10 feet apart 

Channel or Profile line 







Map
of the

MISSISSIPPI RIVER

at
Thousand Islands

showing Wing Dams

built under the direction of P. V. Farguhar, Major of Engineers U.S.A.

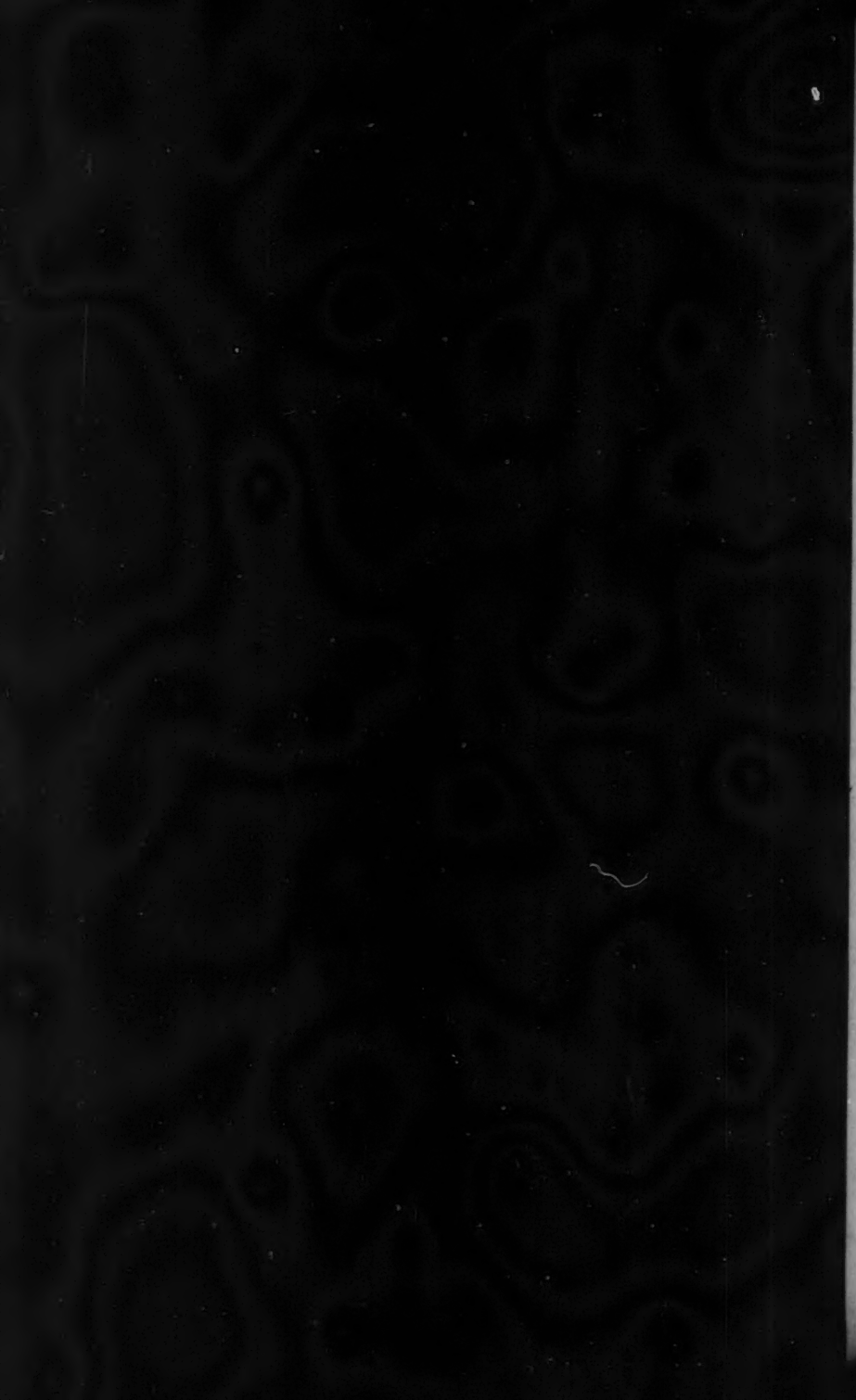
1874.

LOWER STEAMBOAT
— LANDING —



J. H. H. DEL.





AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CXLIX.

RELATIVE QUANTITIES OF MATERIAL IN BRIDGES OF DIFFERENT KINDS, OF VARIOUS HEIGHTS.

II.

A Paper by CHARLES E. EMERY, M.E., Member of the Society.

§ 37. In the previous paper on this subject in August Transactions* is outlined a method of investigation, in which the volume or weight of each member of a skeleton girder is expressed in terms of the volume of a vertical and of a horizontal member having jointly the same volume as a standard diagonal supporting a given load. The expressions for the several members being in the same form, may be summed together in a general equation of the form $V = sW(\Sigma H + \Sigma t)$, in which W represents the panel load or the total load acting vertically on the standard diagonal; H represents the projected vertical height of said diagonal; t $\left(= \frac{P^2}{H} \right)$, in

* Trans., Vol. VI, page 235.

which P equals the panel length or horizontal projection of the diagonal) represents a factor of the volume of a horizontal member subject to the strain due to the horizontal thrust of the standard diagonal, and s (§ 15-17) represents a co-efficient to transmute the products of units of weight or strain by units of length into the corresponding weight of iron members of the given lengths required to withstand the strains due to the given load. An expression for the volume of an entire girder or any desired portion of same may be obtained simply by writing an expression for each member, on this system, varying the co-efficients if necessary to bring the expression in terms of H and l with respect to the standard diagonal (§ 11) and then summing the co-efficients. When the panel loads producing strains are varied, W in the expression is merged in the co-efficients of H and l . The basis of strain adopted per unit of section in the values of s (§ 17) is 10 000 pounds per square inch (7.031 kil. per sq. mm.), but any desired strain may be provided for any number by correspondingly varying the co-efficients of H and l . The minimum value of the equation is obtained by simply making the terms of H equal to the terms of l (§ 12), and in this manner the proper height of the girder is obtained in terms of the panel length to secure the minimum quantity of material. In this determination, the lengths and angles of the several members, and the strains to which they are severally subjected, both in gross and per unit of section, all have due influence in producing the result.

§ 38. A table is given (§ 24) showing, in connection with sketches of twenty-two elementary skeleton girders of different forms, the most economical heights of the several girders and the quantities of material in each for the most economical height and for a height equal to one panel length (§ 26). [Attention is particularly called to an important typographical error in this table, which, with some further corrections and explanations is set forth in the foot note.*]

* In line 19, column 6 of table, for 10.102 substitute 8.214, and in line 20, same column, for 9.177 substitute 10.102, also in § 15 for s , s_1 , and s_2 , substitute s (s_1 , § 17) and s (s_2 , § 17). In equation near end of § 20, for S substitute s .

Attention is called to the fact that $l = \frac{P^2}{H}$ (§ 11), represents the horizontal thrust of the standard diagonal only when $P = W$ as at first assumed (§ 10). Under other conditions, of course, the horizontal thrust would equal $\frac{PW}{H}$, which put $= l$. The volume of a horizontal member in general then equals $sPl = \frac{sP^2W}{H} = sW(l)$, which corresponds with that developed in another way in § 10.

§ 39. The weights of material given in § 20 and § 26 include that in the two or more girders used in supporting the total load. This should be understood also in relation to the girders hereinafter discussed.

§ 40. It is now proposed to compare the relative quantities of material in longer girders of different kinds and of various heights. For this purpose a single track railroad bridge with a total length of 252 feet (76.81 *meters*) has been selected, divided into 14 panels, each 18 feet long. This panel length is in accordance with modern practice, for girders with two web systems, as it prevents the concentrated loads of two modern coupled locomotives from coming on the same system.

§ 41. It has been decided to determine the strains from the actual weights successively imposed on the panel points by a moving train consisting of three coupled locomotives, of the description set forth in the following table, followed by a train assumed to be equivalent to a uniform load of 1 600 pounds per foot:—

	Weight on Driving Wheels.	Driving Wheel Base.	Average Concen- trated load per Foot.	Total weight with Loaded Tender.	Length of Track Occupied.	Average Load per foot.
	Lbs.	Feet.	Lbs.	Lbs.	Feet.	Lbs.
One Consolidation Locomotive.	88 000	14.75	5 980	150 000	55	2 727.3
One Mogul Locomotive.....	68 000	15.00	4 533	125 000	52	2 403.8
One Passenger Locomotive.....	51 500	8.50	6 060	115 000	52	2 211.5
Total, three Locomotives..	390 000	159	2 456.00
				507 434	252	2 168.89
Maximum load on bridge for three locomotives above and train averaging 1 600 pounds per foot. See § 50.				Ditto Tons.		Av. Panel Load. Tons.
				253.72	252	19.52
				Ditto Tonnes.	Meters.	Av. Panel Load. Tonnes.
				230.17	76.81	17.71

§ 42. It is now generally conceded that the strength of railroad bridges should be based on a load of this general character, but it may appear that the weights assumed in this case are greater than it is necessary to use in ordinary practice. It has, however, been considered that

locomotives of the maximum weight named are quite common on many roads and the tendency is, judging from the past, to increase rather than decrease the weight. Such locomotives are usually, but not always, run at slow speeds, but if the members first receiving strains are proportioned to carry them with the ordinary factor of safety, such factor will be none too high for lighter locomotives running at high speeds. Moreover, in high latitudes it is not uncommon to run whole trains of heavy locomotives behind a snow plow in winter, when the iron is in its worst condition to resist shocks. Bridges calculated for heavy concentrated loads, arranged as above so that the average load is not extreme, are safer for all loads, but particularly the exceptional ones which occasionally offer, such as heavy shafting and machinery, guns for the Government and the like. The plan, moreover, appears to involve no additional cost, the amount of material required in an ordinary Whipple truss proportioned on such basis corresponding very nearly with that in similar trusses calculated by the approximate methods founded on a progressing uniform load with locomotive excess.

§ 43. In Fig. 1 are shown the positions of the wheels of the three locomotives, in relation to the panel points, when the center of the driving wheel base of the leading locomotive is over one point. Across vertical lines beneath are written the known weights carried on each pair of locomotive wheels, and the estimated loads carried by the wheels of the tender, the lengths of the lines being also proportioned to the weights so as to form a diagram. On the basis that the track stringers are practically jointed to the floor beams at the panel points, the reactions of the floor beams, considered as abutments, have been ascertained for the actual weights carried on the wheels, acting at the distances from the

Index.	<i>R.</i> Live Load, per Panel.	Index.	<i>R.</i> Live Load, per Panel.	Index.	<i>R.</i> Live Load, per Panel.
	tons.		tons.		tons.
1	10.65	6	20.30	11	14.40
2	33.97	7	17.06	12	14.40
3	19.40	8	24.26	13	14.40
4	22.02	9	18.79		
5	25.50	10	14.82		

panel points shown. The reactions obtained in this way are written on each side of the vertical lines through panel points, and their magni-

tude represented by laying off vertical distances corresponding thereto, which are connected by inclined lines. The sums of the reactions represented by the broken line, form the actual panel live loads, which are designated R , and with indices numbered from the head of the train, are shown in table on preceding page.

§ 44. It is evident from an inspection of the table that as the train moves forward, each member carrying but one panel load must be proportioned for the concentrated load of 33.97 tons (R_2 in table), due to weight on driving wheels of the heaviest locomotive, and that the maximum strain on every member transferring more than one panel load will in general be reached when the load R_2 is transferred through such member. The influence of this principal concentrated load on the section of the members will, however, be less and less as the number of lesser loads combined therewith is increased, hence all the members of short bridges will be proportionally heavier than those of longer ones*, and it will be seen that by using the actual loads derived from a train, arranged as above, the necessary sections to secure uniformity of strength in bridges of any length can be readily and accurately developed.

§ 45. Fig. 2, represents an ordinary Whipple truss with double intersections (two web systems) and of dimensions as previously set forth (§ 40). A and B represent respectively the right and left abutments and the reactions of the same. The loads are supposed to be carried at the bottom of the bridge. The panel points of this and the other girders are numbered from abutment A towards B , and these numbers serve as the indices to all members and quantities relating to the panel points. For example R_1, R_2 , &c., are the live loads derived from table § 43, but with indices changed to correspond with position of train in relation to the junctions; T_1, T_2 , &c., represent the dead loads or weights of structure carried at the panel points, which are assumed to be uniform; W_1, W_2 , &c. ($W = R + T$), represent the total loads at the corresponding junctions; S_5, S_6 , &c., represent the shearing strains at the junctions which are usually referred to abutment B , and are derived from summations of the terms of W . D_5, D_6 , &c., refer to the diagonals starting upward from the junctions of the same numbers and trending towards B . H_7, H_8 , &c., are corresponding verticals. P , which in general represents the panel length, is specially used with an index, thus P_1, P_2 , &c., to represent

* This fact is clearly shown in another way in the paper of Messrs. Clark and Griffin, Trans., Vol. II, p. 93.

one panel length of upper chord, numbering from abutment B , and p_1 , p_2 , &c., refer similarly to the sections of lower chord.

§ 46. The train is supposed to advance from A towards B . If n = no. junctions; $n + 1$ = no. panels; m = no. panels from A to head of train or junction carrying R_1 , § 43; and M_1 , M_2 , &c., represent moments of the loads from abutment A , in terms of the number of panels, we have

$$B = \frac{nT}{2} + \frac{\sum^A m MR}{n+1}, \text{ or the reaction of abutment } B \text{ equals one half the}$$

permanent load of bridge, plus the sums of the moments of the live loads from abutment A , the latter divided by the number of panels. From the reactions for the several positions of the load the corresponding shearing strains are readily obtained, the maximum for any particular panel point occurring usually when the load R_2 , § 43, is over the same.

§ 47. In table opposite page 282, are shown the maximum shearing strains and moments for a Whipple truss, under conditions stated in headings, in connection with the assumed permanent loads, and those obtained by calculation for various heights of girder. In col. 1, the strains are deduced from the actual panel loads as per § 43, and an assumed permanent load of 14 tons per junction, equivalent to 1 555.56 pounds per foot (line 18). The moments correspond to a height of 36 feet. The correct permanent load on basis adopted, having been found to be 1 596.83 pounds per foot (line 19), or 14.37 tons per junction, the shearing strains and moments are shown in column 3, corrected on the basis of a uniformly distributed load, equal to the difference between the assumed and actual permanent loads. In columns 2 and 4 are shown corresponding maximum shearing strains and moments based on an advancing uniformly distributed live load equal, when bridge is fully covered to the maximum total live load, as per cols. 1 and 3 and § 43. Cols. 7 and 8 correspond to cols. 3 and 4 as to nature of loads, but are calculated for a single instead of a double web system.

§ 48. Comparing cols. 3 and 4, it is seen that members proportioned for the uniform progressing load, col. 4, should be increased from 10 to 47 per cent., according to position, to carry the concentrated loads referred to in column 3, and members proportioned to the moments resulting from the given total load when uniformly distributed as per column 4, would also, as shown, be considerably too light for the same total load, arranged upon the basis referred to in column 3. Corresponding results varying in degree will be observed by a comparison of columns 7 and 8.

Maximum Shearing Strains and Moments at different heights for the Whipple type, 252 feet (76.81 meters) long, with 14 panels calculated by different methods for a maximum moving load of 253.72 tons (of 2,000 lbs.) (230.17 tonnes), distributed in different ways.

Based on strains in tension of 10,000 lbs. per sq. in. (7,031 kil. per sq. mm.); in compression; on chords .8 — on end posts .65 — on other posts .45 of the same.

[illegible]

	When $H = 36$	Moments, Tons.	Moments, Tons.	Moments, Tons.	Moments, Tons.	Moments, Tons.	Moments, Tons.	Moments, Tons.
8	S_{11}	118.29	97.01	119.40	98.90	113.35	115.69	211.41
9	S_{12}	233.25	211.71	235.66	213.86	228.31	229.63	246.31
10	P_1	116.62	105.85	117.83	106.92	106.92	106.92	121.15
11	P_2	171.65	154.70	173.41	156.27	156.27	156.27	224.74
12	P_3	253.72	236.13	256.41	238.53	238.53	238.53	305.45
13	P_4	324.04	301.37	327.47	303.83	303.83	303.83	368.46
14	P_5	372.71	350.12	376.70	353.18	353.18	353.18	412.46
15	P_6	408.00	382.69	412.36	386.08	386.08	386.08	437.92
16	P_7	419.62	398.98	424.17	402.54	402.54	402.54	444.23
17	P_8	419.62	398.98	424.17	402.54	402.54	402.54	444.23
18	Assumed permanent load per foot, lbs.	1555.56	1450.00	1555.56	1450.00	1450.00	1555.56	1800.00
19	Actual permanent load, when $H = 36$ feet, lbs.	1596.83	1486.83	1486.83	1778.42
20	Actual permanent load per panel, tons.	14.37	13.38	13.38	16.01
21	Most economical height of girder on basis of strains adopted.	47.97 ft.	50.52 ft.	36.92 ft.
22	When $H = 60$ At economical height, nominal height, line 21.	927.05	793.07 lbs.	1327.5 lbs.
23	When $H = 36$	897.09	778.29 "	1128.00 "
24	When $H = 24$	946.83	836.83 "	1128.42 "
	Weights of girders per foot $\times \frac{1}{2}$ lbs. (add 650 for total permanent load.)	*1209.35	11087.00 "	1282.50 "

* Corrected from an assumed permanent load of 1700 lbs. per foot. † Ass. per. load 1650 lbs. per foot. ‡ Ass. per. load 1761.11 lbs. per foot.

§ 49. In column 5 are shown shearing strains obtained by adding to each of those due to a uniform progressing load (column 4), the maximum "locomotive excess" or the difference between the largest panel load R_2 , § 43, and the average panel load, and equals $(33.97 - 19.52 =)$ 14.45 tons in this case. Members proportioned by column 5, would be safe as compared with column 3, but some would be heavier than necessary and others lighter. In column 6, are shown shearing strains calculated for the same total live load as in previous cases, but distributed in groups of different intensities, viz., first, one panel load equal to the maximum T_2 , § 43, followed by uniform panel loads equal in the aggregate, including that above named, to the weight of the three locomotives and distributed in the same length,—the latter followed as before by a train averaging 1600 pounds per foot. Comparing columns 1 and 6 (the latter not having been corrected), it will be seen that even on this basis the shearing strains in latter column vary from those produced by actual loads, but the approximation is better than in column 5, and the method would answer pretty well for a single web system under all conditions, and for a multiple web system when the panel lengths were arranged so that the concentrated weights on driving wheels of two adjacent locomotives were not supported by the same system.

§ 50. On the whole, the more accurate method, in all cases, is to use, in calculation actual loads derived from a certain standard train of locomotives and cars. The strain for the Whipple truss, referred to in column 3 of table, and for the other girders hereafter compared therewith, are calculated on the basis of the panel loads set forth in § 43. The maximum load of 253.72 tons derived therefrom occurs when $m = 14$, that is when the load R_2 , § 43, runs off the bridge proper over the farther abutment, and a greater panel load comes on the other end of bridge from the following train. The maximum moments occur for the two center panels P_6, P_7 , when $m = 12$, that is when the head of train is at junction 12 and the concentrated load T_2 , § 43, from the driving wheels of the first locomotive is over junction No. 11, or 54 feet from the end of the bridge. The maximum moments for the remainder of upper chord occur when train is advanced to the next panel (No. 13). The two end portions of lower chord, p_1, p_2 , alone receive their maximum strain when the bridge is supporting the maximum load. For uniform loading as per column 4, table, the maximum moments at all points are obtained, as is well known, when bridge is fully loaded.

§ 51. The expression for the volume of a half span of the Whipple girder, referred to in column 3 of table, under conditions as to strains per unit of section set forth in general headings, is as follows :

$$.5 Vg = S (1\,555.25 H + 11\,046.39 t).$$

Putting terms of H = terms of t , § 13, the economical height is found to be 47.97 feet (line 21, table.) Solving the equation for $H=36$, $t = \frac{P^2}{H} = 9$, and using s_2 , § 17, as the value of s , the weight per foot of girder, after adding one-eighth for connections, will be found to be 936.44 pounds. Adding to this 350 pounds per foot to cover the permanent weight of iron in stringers, floor beams and the lateral and wind braces in the floor and roof, and 300 pounds per foot for weight of track, the calculated permanent load becomes 1 586.44 pounds per foot. The assumed permanent load being 1 555.56 pounds per foot, the result above is too small by the amount of metal required to support the additional permanent load. Making this correction, the corrected weight of girder increased one-eighth for connections is found to be 946.83 pounds (line 23, table), and corrected permanent load 650 pounds more, or 1 596.83 pounds (line 19, table).

§ 52. In a similar manner, the corrected weight of girder (both sides included), for different heights, will be found to be : for 60 feet, 927.05 pounds per foot ; for 47.97 feet, the economical height, 897.09 pounds ; for 36 feet, 946.83 pounds, and for 24 feet, 1 209.35 pounds. These weights are not exactly correct, except when the corrected weight of girder, plus 650, equals the assumed load, but for trussed girders the difference is very small and the equations are practically correct within considerable limits. For instance, by calculating column 4 with an assumed load of 1 650 pounds per foot, the corrected permanent load was found to be 1 481.8 pounds ; yet with an assumed permanent load, 200 pounds less per foot (line 18), the actual or corrected permanent load was found to be 1 486.83 pounds (line 19), a difference of only one-third of one per cent. In arch girders the equations are correct only when the assumed and actual permanent loads are practically identical.

§ 53. The weights of girders at different heights above referred to are based on the condition that the strain on vertical compression members is in all cases 4 500 pounds per square inch. The economical height on this basis is so great that it would require that extra vertical and horizon-

tal bracing be put in to secure the stability of the columns and counteract the increased overturning moment due to wind pressure. This bracing would support the posts so near the centre that each portion would be shorter than the whole length of columns used ordinarily so that the cross sections could be diminished a little, which fact, with the saving in chord sections due to the increased height, would more than compensate, in mere weight of material, for the extra bracing. The price per pound would, however, be increased somewhat, and any increase of height requiring that the posts be each made in two pieces would involve details of an objectionable and expensive character, so that the minimum cost, rather than weight, can be determined only by considering the details and the facilities of manufacture.

‡ 54. The weights of girders per foot at different heights, and for different conditions have been plotted in Fig. 7, and curves drawn to show the relations to the eye.

‡ 55. In the following table is shown a comparison of several different forms of girder each 252 feet (76.81 m.) long, and calculated for the same load used for the Whipple girder in column 3 of table, opposite page 282. The designations will be readily understood in connection with the figures referred to.

No. of line.	DESIGNATION OF GIRDER.	1	2	3	4	5	6	7	8 *
		Eco- nomical Height on basis of Strains adopted	WEIGHT OF GIRDER per foot (both sides) $\times 1\frac{1}{2}$, for—			Total Weight of Iron per foot.	Perma- nent load per foot.	Perma- nent load per meters.	
			Height 60 feet.	Eco- nomical Height.	Height 36 feet.	Height 24 feet.			
			Feet.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Kilogr.
1	Whipple Truss... Fig. 2.	47.97	927.05	897.09	946.83	1209.35	1296.83	1396.83	2376.4
2	Triangular Truss.. Fig. 3.	43.71	1021.19	955.32	979.62	1207.94	1329.62	1629.62	2425.2
3	Warren Truss..... Fig. 4.	44.89	955.65	904.32	933.74	1162.98	1283.74	1583.74	2356.9
4	Bow String Girder Fig. 5.	Approx. 56	1116.32	1466.32	1766.32	2628.6
5	Suspended Arch.. Fig. 6.	Approx. 53	1178.60	1528.60	1828.60	2721.3

The above results are based on strains in tension of 10 000. pounds per square inch (7.031 kil, pr. sq. m m.). In compression: on chords and

* For col. 4, subtract 967.32 kilogrammes; for col. 6, subtract 297.64 kilogrammes.

the rib of the bow string girder (line 4) .8 the above ; on all the columns of web, vertical or inclined, .45 of the same, except end posts which are based on .65 of the above. The arches are parabolic. For the suspended arch the loads are assumed to be carried above the girder.

§ 56. An examination of the previous table shows the remarkable fact that the amount of material required for each of the three trusses named is substantially the same. For a height of 36 feet the Warren girder appears in table to be slightly the most economical, but would have no advantage over the Whipple in practice, as doubtless some material would need to be provided, in the design, to stiffen the long sections of upper chord. The verticals of the Warren girder are necessarily proportioned for the concentrated load, but the loss due to this is more than balanced by the fact that there is but one web system, so that for the heavier members the maximum strains are derived from the sum of the concentrated load with a number of lighter loads, giving a lower average than for two web systems, as in the triangular truss line 3. The latter form, doubtless, has its advantages in riveted girders made of "shape iron," as mutual support may be readily obtained at the crossings of the web members.

§ 57. In calculating the bow string girder, an assumed permanent load of 1 800 pounds per foot gave 58.804 feet as the economical height, and 1 586.53 lbs per foot as the corrected permanent load derived from weight of girder at that height, and an assumed load of 1 555.56 pounds per foot gave 50.07 feet as the economical height, and 1 862.48 pounds as the permanent load derived from the weight of girder. This shows plainly how rapidly the strains on the bracing are reduced as the permanent load is increased, and *vice versa*. With the permanent load first named, the calculated permanent load for the height of 33.5 feet is 1 800.61 pounds, or practically the same as assumed, the weight of girder proper being 1 150.61 pounds. The weight per foot of a Whipple girder, 33.5 feet height, being 957.8 pounds, and at 36 feet height 946.83 pounds, the relative weights of the two girders are assumed to be the same at both heights, giving 1 116.32 pounds as the weight per foot of a bow string girder 36 feet high, as written in table. The corresponding weight of the suspended arch was obtained in a similar manner, instead of making more trials. The results being correct at one height, those de-

rived therefrom are sufficiently accurate for comparison at a height of 36 feet. Evidently, both the bow string and suspended arch girders require considerable more material to support movable loads, particularly concentrated loads, than the plain trusses, which agrees with the experience of some of the oldest bridge builders in the country.

§ 58. For the purpose of comparison ultimately with the hinged arch, and as a matter of interest, the maximum strains on the braces for the bow string girder 33.5 feet high are given in the following table. The quantities represent the vertical loads transferred, designated $y_1, y_2, \&c.$, the indices being as per § 45.

Index.	y . Tons.	Index.	y . Tons.	Index.	y . Tons.
1	39.59	5	39.92	9	118.68
2	62.05	6	78.14	10	107.53
3	70.00	7	104.02	11	84.03
4	48.87	8	117.55	12	49.25

§ 59. The train is supposed to move from A towards B , fig. 5, and the braces are strained to transfer part of the load in the same direction. The maximum values of y for the first four junctions occur where the half span is about half loaded, that is, when the head of train has reached J_4 . The remainder of the strains reach a maximum when the first six of the thirteen junctions are loaded, except y_{12} , which is a little higher when head of the train reaches junction 8. These strains were derived from the simple consideration that an arch rib composed of straight sections or members, assumed to be jointed together, will support, without straining web members, loads severally proportioned to the deflections of the rib sections. Loads not arranged in this way are redistributed by the bracing until equilibrium on above basis is obtained. Summations for loads at successive junctions are made from the point where moments are a maximum, each way to the abutments.

§ 60. The distribution of material in the Whipple truss line 1, and bow string girder line 4, table § 55, is shown by the following table, calculated for a height of 33.5 feet :

No. of Line.		Whipple Truss Girder, Average Weights per foot. $\times 1\frac{1}{4}$ Lbs.	Bow String Girder, Average Weights per foot. $\times 1\frac{1}{4}$ Lbs.
1	Upper Chord.....	326.28
2	Arch Rib.....	492.50
3	Lower Chord.....	209.44	359.64
4	Web.....	440.18	298.47
	Totals.....	975.90	1 150.61

In all forms of arch girder the weight of the rib or arch proper, line 2, would remain the same for the same system of loading, and any saving of material would require to be effected in the web members and by the use of an earth chord.

The subject of hinged and other arches, and of the continuous girder, is deferred for future consideration.



Plate XX

Fig. 2.
Whipple Truss.

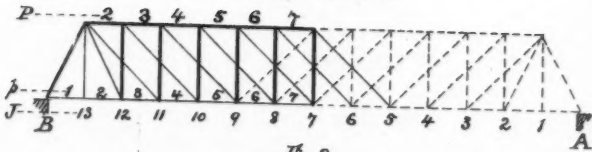


Fig. 3
Triangular Truss.

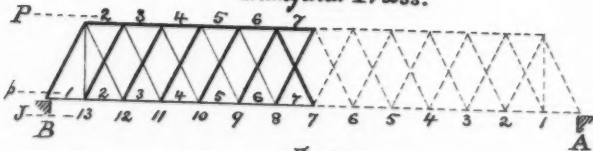


Fig. 4.
Warren Girder.

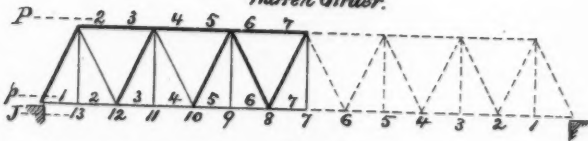


Fig. 5
Bow String Girder.



Fig. 6.
Suspended Arch.

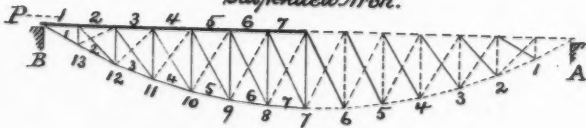
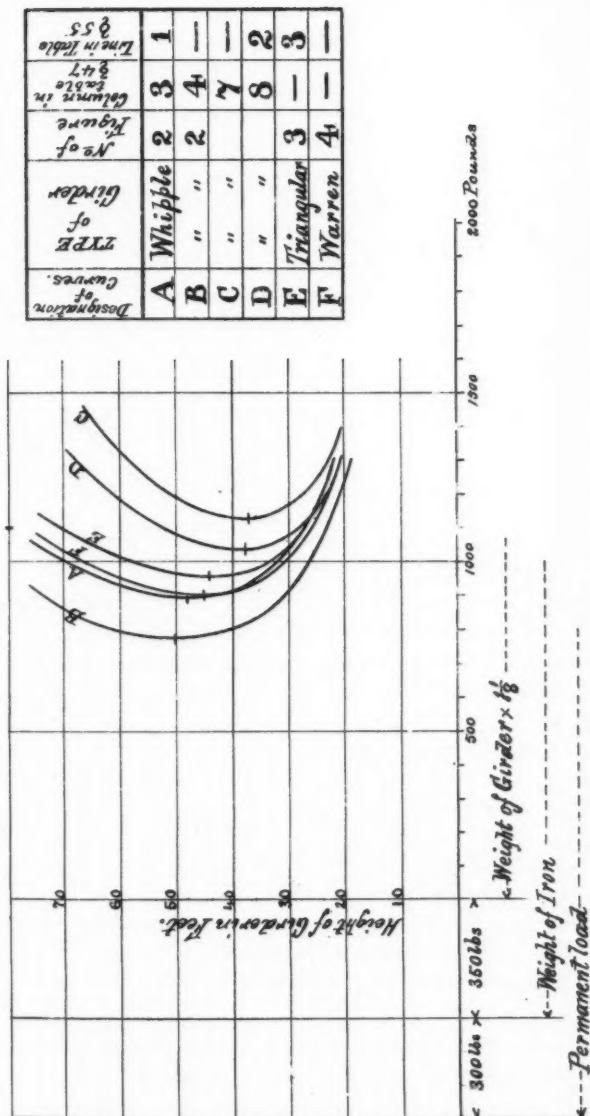




Fig. 7.

Curves showing relative weights of Girders 252 feet long at different heights





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CL.

DESCRIPTION OF SURVEY FOR DETERMINING THE SLOPE OF WATER SURFACE IN THE ERIE CANAL.

A Paper by W. H. SEARLES, C. E., Member of the Society.

READ AT THE ANNUAL CONVENTION, APRIL 24TH, 1877.

The importance of obtaining levels of extreme accuracy in any case in which the natural flow of water in rivers or canals is concerned is obvious, but especially where it is desired to ascertain the descent of the surface grade at low velocities. I have had occasion during the past year to direct the establishment of a series of benches on masonry along the line of the Erie Canal (Western Division), which undertaking has afforded some points of interest.

The Erie Canal from Buffalo to Montezuma, a distance of about 150 miles, depends almost entirely on Lake Erie for its supply of water. In this distance are found "levels" respectively 12, 17, 26 and 62½ miles in length, Besides many shorter ones. It is evident that the surfaces of these so-called "levels" must have a certain amount of fall to induce the water to flow through them. To ascertain this fall, and to establish permanent water-marks at various points along the line, were among the objects of the survey under my direction.

There were no benches remaining from the surveys connected with the construction or enlargement of the canal, so that to all intents and

purposes the survey was an original one. The levels were run by a single leveller, Mr. Daniel Bontecou, of New York, and a single rodman, Mr. Albert A. Simpson, of Poughkeepsie, using a 16-inch Stackpole level and a New York target rod. Care was taken to make the distances of back-sights and fore-sights equal, but the turning points were not preserved. Benches were established at the rate of one to each half mile on an average. As others in the party were engaged in taking cross-sections of the canal by sounding, the levelling proceeded slowly, and ample time was afforded to obtain accurate readings.

The test levels were proceeded with more rapidly. They were run in part by the same leveller and rodman, and partly by others, but in all cases with great care. The results show very satisfactorily, compared with the results of the best and most elaborate surveys in this or other countries. The discrepancies in elevation were always small, while the discrepancy in the rise or fall between any two consecutive benches was trifling. In a few cases, however, where the variation appeared too great to pass unnoticed, the ground between the two benches was gone over a third or even a fourth time, and, if necessary, the levels of the original line were corrected by the latest running. The agreement of the original survey and the test was thus made still nearer. Neither set of levels, however, was adopted as correct.

A table of the rise and fall from each bench to the next was prepared, showing all the results obtained. After striking out a very few that were obviously in error, a mean of the remainder was taken as the most probable value of the rise or fall in each case. By adding or subtracting these figures successively to the elevation of the first bench, the probable elevations of all the other benches were obtained, the probable error in rise or fall was calculated by the method of least squares, while the probable error in elevation was found by Mr. Airy's rule $e\sqrt{m}$, in which e = the probable average error per mile, and m the number of miles. The table herewith presented shows at one view the results of these levels and calculations. The average error per mile is found to be $\pm.0088$ ft., while the probable error in the terminal bench, in a distance of 136 miles, is $\pm.103$ ft.

Those who desire to compare these results with foreign work of the best class are referred to the paper of Wilfred Airy, B. A. M. Inst. C. E., quoted in the *Engineering News*, March 31st, 1877, and following numbers.

COMPARISON OF SURVEY LEVELS WITH TEST LEVELS.

Section of 34 Miles.	Total dist. Miles.	Rise per Section.	Total Rise.	Divergence per Section.	Terminal divergence.	Greatest divergence in any one mile.	Average divergence per mile.	Average error per mile.	Probable error in terminal bench.
I.	34	66.482	66.482	+.116	+.116	.076	.0236	.0118	±
II.	68	47.001	113.483	-.083	+.033	.112	.0245	.0123	.069
III.	102	0.406	113.889	-.373	-.340	.085	.0292	.0146	.130
IV.	136	62.842	176.731	+.247	-.093	.078	.0235	.0117	.147

General average error per mile..... ±.0126

COMPARISON OF CORRECTED SURVEY LEVELS WITH TEST LEVELS.

Section.	Total distance miles	Divergence per Section.	Terminal divergence.	Greatest divergence in one mile.	Average divergence per mile.	Average error per mile.	Probable error in terminal bench.
I.	34	-.069	-.069	.076	.0236	.0118	±
II.	68	+.044	-.025	.048	.0147	.0073	.078
III.	102	-.015	-.040	.076	.0193	.0096	.097
IV.	136	+.044	+.004	.045	.0128	.0064	.103

General average error per mile..... ±.0088

Level made by Stackpole, New York. Length, 16 inches; magnifying power, 16 diameters; dip of telescope per $\frac{1}{10}$ th-inch run of bubble = 11 seconds; "New York" Rod graduated to $\frac{1}{100}$ th foot, with target reading to $\frac{1}{1000}$ ths.

On Surrey.—Daniel Bontecon, leveller; Albert A. Simpson, rodman.

On Test.—The same, and also Robert P. Staats, leveller; William B. Maxwell, rodman.

The surface of the water was observed by the leveller every time a cross-section was taken, or once in every 132 feet. The resulting profile showed an undulating line, caused by the irregularity of feed and lock-ages, and on the larger levels by the action of the wind. There was, moreover, a scarcity of water during a portion of the time that the survey was in progress. It would not answer, therefore, to select even the average line of this profile as the standard water surface, and it was decided to obtain the latter by calculation.

For this purpose the "level" or grade of 62½ miles, between Lockport and Rochester, was divided into ten lengths of 6½ miles each, and the calculation for each made separately.

The data given are :

The *depth of water* at the beginning of a length upon the assumed grade line at the bottom ;

The *area of the waterway*, and the *wet perimeter* due to that depth—both of which were determined by plotting on a large scale all the cross-sections taken on the survey ;

And the *required average volume of discharge* through the length in question in a given time.

The average or mean sectional area of a given length of canal was obtained in the usual manner, by dividing the aggregate area of the cross-sections by their number.

To obtain the value of the mean wet perimeter, a number of sections were selected whose mean area was equal to the mean area of all the sections on the length of canal considered.

The wet perimeter of each was measured by scale, and the area of each section was divided by the wet perimeter so found ; giving the mean radius or ratio of each section.

A mean of these ratios was then found, and the mean area divided by this gave the mean wet perimeter required.

The mean volume of discharge was assumed to satisfy the requirements of navigation, after supplying all losses due to evaporation, filtration and properly constructed waste-weirs.

The formula adopted was the well known one given by Eytelwein :

$$F = .000111415 \frac{P}{A} v^2 + .000024647 \frac{P}{A} v.$$

in which F = fall of surface in feet per foot,

and v = mean velocity in feet per second,

p = wet perimeter in feet,

and A = area of water section in square feet.

But, for convenience of calculation, the formula was first reduced to the following equivalent form :

$$f = p \frac{(V + 6.534)^2}{6119.6 A}.$$

in which f = fall of surface in feet per mile.

V = mean velocity in feet per minute = $\frac{D}{A}$

and D = discharge in cubic feet per minute.

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TABLE OF GRADES AND SURFACE DESCENT,
SHOWING DISCHARGE IN PRISM AND LOSS OF WATER AT SEVERAL POINTS.

LOCALITY.	Distance, Miles.	Depth on Grade line.	Fall of Surf. per Mile.	Total Fall of Surface	Ave. Surf. Width.	Ave. Area in sq. ft.	Average Perimeter	Average Ratio.	Ave. Mean Velocity. ft. per Min. V.	Average Discharge. Cu. ft. per Min. D.	Loss of Water. Cu. ft. per Min.
	<i>L.</i>	<i>d.</i>	<i>f.</i>	<i>F.</i>	<i>fl.</i>	<i>A.</i>	<i>p.</i>	<i>R.</i>			
Lockport.....	8.000	33 755.
	6.24	7.937	.068	.425	96.45	693.68	105.89	6.5311	45.678	31 686.	2069.
Gasport.....	7.875	31 092.	594.
	6.25	7.825	.064	.400	97.70	680.09	101.05	6.7303	44.807	30 474.	618.
Middleport.....	7.775	28 100.	2374.
	6.25	7.737	.060	.375	93.38	642.14	96.74	6.6377	42.834	27 506.	594.
Medina.....	7.700	25 867.	1639.
	6.25	7.662	.060	.375	90.76	603.04	94.357	6.3912	41.908	25 273.	594.
Eagle Harbor.....	7.625	24 154.	1119.
	6.25	7.600	.056	.350	87.96	572.91	91.66	6.2504	39.748	22 772.	1382.
Hindsburgh.....	7.575	22 178.	594.
	6.25	7.575	.048	.300	84.85	575.98	89.10	6.4644	37.042	21 335.	843.
Holley.....	7.575	19 024.	2311.
	6.25	7.575	.048	.300	80.95	514.68	84.32	6.1039	35.809	18 430.	594.
Cooley's Basin.....	7.575	17 228.	1142.
	6.25	7.600	.040	.250	82.22	511.41	83.43	6.1299	32.202	16 468.	820.
Spencerport.....	7.625	15 874.	594.
	6.25	7.662	.036	.225	81.01	518.77	84.86	6.1133	30.164	15 280.	594.
Wide Water.....	7.700	14 686.	594.
	6.25	7.762	.028	.165	73.50	509.38	77.81	6.5464	26.958	14 092.	594.
Rochester.....	7.835	12 175.	1917.

Fall of Grade line, per mile..... .048

Total Fall of Grade..... 3.000

Total Fall of Surface..... 3.165

The rate of descent (f) having been thus ascertained for a given average discharge, the discharge at the upper end was assumed to be enough greater than the average, and the discharge at the lower end enough less, to provide for the estimated losses by the way on the length of canal considered. While not strictly correct, this method is as close as the given data in the case, and was found to answer every practical purpose.

The discharge of water on the second length evidently depends on what quantity can be received from the first, there being no other source of supply; the depth of water at the upper end of the second must be identical with the depth at the lower end of the first; and upon this depth the areas and wet perimeter depend. Using the data thus limited, the rate of descent of the second length was found; and so on, in succession, for each of the ten (10) lengths. In some cases, the descent of the surface not being parallel to the bottom grade, modified the value of the mean area and perimeter to such an extent that it became necessary to use the corrected values in obtaining the rate of descent by a second approximation.

The theoretical water surface thus determined was made the standard for the water of the canal, and permanent water marks, or "targets," were cut and painted upon the masonry at intervals throughout the 62½ miles to correspond to the heights thus calculated.

The profile of the theoretical surface is a smooth and gently curving line; yet it conforms quite well, on the average, to the irregularly undulating profile of the water surface as surveyed.

After regulating the height of the spillways to correspond to the heights assumed for them in the calculation, the water of the canal was found to coincide closely with the water marks established—subject, of course, to the ordinary fluctuations—thus confirming the correctness of the formula and the accuracy of the calculations.

The actual cross-sections vary widely from a standard section, and from each other, in shape and area; hence the care taken in preparing the mean area and perimeter was necessary. The amount of waste allowed for evaporation and filtration is 190 cubic feet per mile per minute.

The table here submitted will show at a glance the actual quantities entering into the calculation, and may repay for perusal those specially interested in this subject.

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NOTE.—The Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

DISCUSSION OF

CONNECTED-ARC MARINE BOILERS.*

Mr. J. F. FLAGG : The paper by Mr. Charles E. Emery on Connected-Arc Marine Boilers has been read by me with much interest and pleasure. The plan developed is a novel one, to me at least, and seems to be a great improvement for the cramped boiler room generally available on sea-going steamers.

It strikes me that another point might be added, suggested by the last paragraph on p. 172 (above the foot note), viz., concerning the *stability of the equilibrium* in the connected boilers. Inasmuch as "the arc of larger radius tends to straighten (increase the radius of) the arc of lesser radius," if from any cause, whether from inaccuracy of manufacture, or from subsequent accidental distortion, the different arcs should vary in curvature, the effect of steam pressure would be, not to increase distortion, but to restore them to, and keep them in their normal shape, as would occur in fact in a plain cylindrical form.

* Referring to No. CXLI. Vol. 6, page 169.

With regard, however, to the form indicated in Figure 9* (a minor matter in the paper), there seems to be an error in the resolution of forces. In the force triangle AEF , the force acting in direction BE , represented by AE , is resolved into the force EF taken up by the strut AB , and into the force AF acting at right angles to AB . This last force is not accounted for, and from this and other similar decompositions we would have, besides the forces taken up by the struts, residual forces (as in figure I. below), which are not opposite each other, and cannot be taken up directly by the shell at A , B and C .

Fig. I.

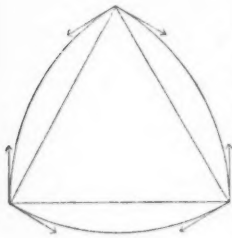


Fig. II.

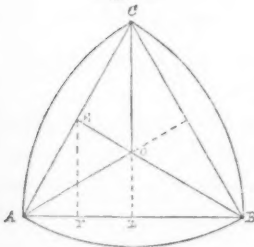
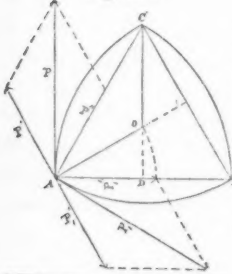


Fig. III.



Bisecting the angle CAB (see figure II.) by the line AO , the steam pressure upon the arc AC acting at A in the direction BE , and represented by AE in the force triangle AEO , should be divided into EO taken up by the strut AC , and AO acting at right angles to AO ; the latter being equal and opposite to a similar component of the force upon AB acting at A , is taken up by the shell at A .

The strain upon the strut then becomes

$$BE - EO = BO = CO = \frac{2}{3} CD$$

whilst in the paper it is deduced as being

$$CD - DG = CG = \frac{1}{2} CD, \text{ less in amount.}$$

The strains may be more directly deduced as follows (see figure III.) :

The total tangential pressure at A produced by the steam upon arc AC is P , at right angles to AB —made equal to AB (the radius of arc) in amount—and may be decomposed at P' at right angles to AO , and P' in direction of strut AC ; in like manner tangential force P_1 , due to pressure upon AB , may be decomposed into P_1' and P_1'' . P' and P_1' are equal and opposite, and determine the strain upon shell at A ; $P'' = P_1''$ are the total strains upon struts AB and AC . Or, forming the force triangle ACO , AC represents the

tangential force P_1 , $CO = \frac{2}{3} CD$ (as before) represents the strain on struts, and AO the strain taken up by shell at A .

ERRATA.

Transactions, Vol. VI, No. CXLVII. Paper by C. Shaler Smith on Proportions of Eye-Bar Heads and Pins, as determined by experiments. Page 264, line 16, for *Welded* read *Weldless*. Page 267, line 3, for *Welded* read *Weldless*. Page 266, in heading of table, for *Welded* read *Weldless*. Page 265, line 4, for *presumed to the same* read *presumed to be the same*.

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CL.

NOMENCLATURE OF BUILDING STONES AND OF STONE MASONRY.*

A Paper by J. JAMES R. CROES, WILLIAM E. MERRILL and EDGAR
B. VAN WINKLE, Members of the Society.

PRESENTED AT THE ANNUAL MEETING, NOVEMBER 7TH, 1877.†

Building stones are classified either in accordance with the manner in which they are individually prepared for use, or in accordance with the manner in which they are aggregated into masonry. The latter classification is in some measure dependent upon the former, as the nomenclature of masonry varies, not only with the combination, but also with the character of the work on the individual stone. The first named classification is that of the stone-cutter, and the other that of the mason.

The object of this paper is to discuss building stones from these two stand points, with the view of obtaining a uniform nomenclature. The use of the same terms throughout the United States would prevent misunderstandings and make engineering practice more uniform, and better understood when records of completed work are examined. The present diversity of nomenclature is unsystematic and objectionable on many grounds.

* Proceedings, Vol. I, p. 210. † Proceedings, November, 1877, Vol. III, p. 111.

TOOLS USED IN STONE-CUTTING.

In order to describe intelligibly the various kinds of building stones, it will be necessary to begin with a description of the tools used in stone-cutting, as the names of many kinds of dressed stones are directly derived from those of the tools used in dressing them.

Formerly stone-cutters' tools were made of iron with steel edges; the modern practice is to make them wholly of steel.

The **DOUBLE FACE HAMMER** (Fig. 1) is a heavy tool weighing from 20 to 30 lbs., used for roughly shaping stones as they come from the quarry and for knocking off projections. This is used only for the roughest work.

The **FACE HAMMER** (Fig. 2) has one blunt, and one cutting end, and is used for the same purpose as the double face hammer, where less weight is required. The cutting end is used for roughly squaring stones preparatory to the use of finer tools.

The **CAVIL** (Fig. 3), has one blunt and one pyramidal or pointed end. It weighs from 15 to 20 lbs. Used in quarries for roughly shaping stone for transportation.

The **PICK** (Fig. 4), somewhat resembles the pick used in digging, and is used for rough dressing, mostly on limestone and sandstone. Its length varies from 15 to 24 inches, the thickness at the eye being about 2 inches.

The **AXE OR PEAN HAMMER** (Fig. 5) has two opposite cutting edges. It is used for making drafts around the arris or edge of stones and in reducing faces and sometimes joints, to a level. Its length is about 10 inches and the cutting edge about 4 inches. It is used after the point and before the patent hammer.

The **TOOTH AXE** (Fig. 6) is like the axe, except that its cutting edges are divided into teeth, the number of which vary with the kind of work required. This tool is not used in granite and gneiss cutting.

The **BUSH HAMMER** (Fig. 7) is a square prism of steel whose ends are cut into a number of pyramidal points. The length of the hammer is from 4 to 8 inches, and the cutting face from 2 to 4 inches square. The points vary in number and in size with the work to be done. One end is sometimes made with a cutting edge like that of the axe.

The **CRANDALL** (Fig. 8) is a malleable iron bar about two feet long, slightly flattened at one end. In this end is a slot, 3 inches long and

$\frac{3}{4}$ -inch wide. Through this slot are passed ten double-headed points of $\frac{1}{4}$ -inch square steel, 9 inches long, which are held in place by a key.

The PATENT HAMMER (Fig. 9) is a double-headed tool so formed as to hold at each end a set of wide thin chisels. The tool is in two parts which are held together by the bolts which hold the chisels. Lateral motion is prevented by four guards on one of the pieces.

The tool without the teeth is $5\frac{1}{2} + 2\frac{3}{4} + 1\frac{1}{2}$ inches. The teeth are $2\frac{3}{4}$ inches wide. Their thickness varies from $\frac{1}{12}$ to $\frac{1}{8}$ of an inch. This tool is used for giving a finish to the surface of stones.

All of the above mentioned are two-handed, or require both hands of the workman to use them.

The remaining tools to be described require the use of only one hand for each.

The HAND HAMMER (Fig. 10), weighing from two to five pounds, is used in drilling holes, and in pointing and chiselling the harder rocks.

The Mallet (Fig. 11) is used where the softer limestones and sandstones are to be cut.

The PITCHING CHISEL (Fig. 12) is usually of $1\frac{1}{2}$ inch, octagonal steel, spread on the cutting end to a rectangle of $\frac{1}{2} + 2\frac{1}{2}$ inches. It is used to make a well defined edge to the face of a stone, a line being marked on the joint surface, to which the chisel is applied, and the portion of the stone outside of the line broken off by a blow with the hand hammer on the head of the chisel.

The POINT (Fig. 13) is made of round or octagonal rods of steel, from $\frac{1}{4}$ inch to 1 inch diameter. It is made about 12 inches long with one end brought to a point. It is used until its length is reduced to about 5 inches. It is employed for dressing off the irregular surface of stones, either for a permanent finish or preparatory to the use of the axe. According to the hardness of the stone, either the hand hammer or mallet is used with it.

The CHISEL (Fig. 14) of round steel of $\frac{1}{2}$ to $\frac{3}{4}$ inch in diameter, and about 10 inches long, with one end brought to a cutting edge from $\frac{1}{4}$ inch to 2 inches wide, is used for cutting drafts or margins on the face of stones.

The TOOTH CHISEL (Fig. 15) is the same as the chisel except that the cutting edge is divided into teeth. It is used only on marbles and sandstones.

The SPLITTING CHISEL (Fig. 16), is used chiefly on the softer stratified stones, and sometimes on fine architectural carvings in granite.

The PLUG, a truncated wedge of steel, and THE FEATHERS, of half-round malleable iron (Fig. 17), are used for splitting unstratified stone. A row of holes is made with the DRILL (fig. 18) on the line on which the fracture is to be made. In each of these holes two feathers are inserted, and the plugs lightly driven in between them. The plugs are then gradually driven home by light blows of the hand hammer on each in succession until the stone splits.

In architectural carving, a variety of chisels of different forms are used. For most of these no specific names exist, and their shapes are varied with the special work to be done.

STONE CUTTING.

All stones used in building come under one of three classes, viz. :—

I.—Rough stones that are used as they come from the quarry.

II.—Stones roughly squared and dressed,

III.—Stones accurately squared and finely dressed.

In practice, the line of separation between them is not very distinctly marked, but one class gradually merges into the next.

I.—UNSQURED STONES OR RUBBLE.—This class covers all stones which are used as they come from the quarry without other preparation than the removal of very acute angles, and excessive projections from the general figure. The term "backing" which is frequently applied to this class of stone is inappropriate, as it properly designates material used in a certain relative position in a wall, whereas, stones of this kind may be used in any position.

II.—SQUARED STONES.—This class covers all stones that are roughly squared and roughly dressed on beds and joints. The dressing is usually done with the face hammer or the axe, or in soft stones with the tooth hammer. In gneiss it may be necessary to use the point sometimes. The distinction between this class and the third, lies in the degree of closeness of the joints which is demanded. Where the dressing on the joints is such that the distance between the general planes of the surfaces of adjoining stones is one-half inch or more, the stones properly belong to this class.

Three subdivisions of this class may be made, depending on the character of the face of the stone.

(a.) *Quarry-faced* stones are those whose faces are left untouched as they come from the quarry.

(b.) *Pitch-faced* stones are those on which the arris is clearly defined by a line beyond which the rock is cut away by the pitching chisel, so as to give edges that are approximately true.

(c.) *Drafted* stones are those on which the face is surrounded by a chisel draft, the space inside the draft being left rough. Ordinarily, however, this is done only on stones in which the cutting of the joints is such as to exclude them from this class.

In ordering stones of this class the specifications should always state the width of the bed and end joints which are expected and how far the surface of the face may project beyond the plane of the edge. In practice the projection varies between 1" and 6". It should also be specified whether or not the faces are to be drafted.

III.—CUT STONES.—This class covers all squared stones with smoothly dressed beds and joints. As a rule all the edges of cut stones are drafted, and between the drafts the stone is smoothly dressed. The face, however, is often left rough, when the constructions are massive.

In architecture there are a great many ways in which the faces of cut stone may be dressed, but the following are those that will usually be met in engineering work:

Rough Pointed.—When it is necessary to remove an inch or more from the face of a stone, it is done by the pick or heavy points until the projections vary from $\frac{1}{2}$ " to 1". The stone is then said to be rough pointed. This

operation precedes all others in dressing limestone and granite.

Fine Pointed.—If a smoother finish is desired, rough pointing is followed by fine pointing, which is done with a fine point. It is only used where the finish made by it is to be final, and never as a preparation for final finish by another tool.

Fig. 19.

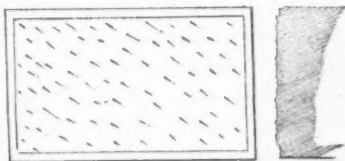
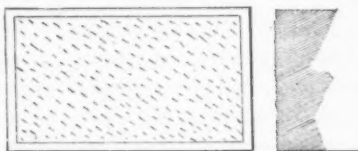
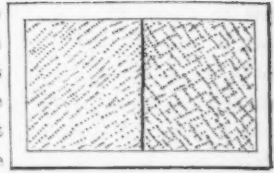


Fig. 20.



Crandalled.—This is only a speedy method of pointing, the effect being the same as fine pointing, except that the dots on the stone are more regular. The variations of level are about $\frac{1}{4}$ ", and the rows are made parallel. When other rows, at right angles to the first, are introduced, the stone is said to be *cross-crandalled*.

Fig. 21.



Axed or Pean Hammered, and Patent Hammered.—These two vary only in the degree of smoothness of the surface which is produced.

Fig. 22.

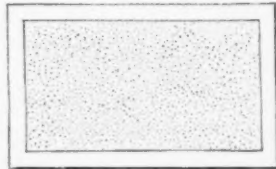


The number of blades in a patent hammer varies from 6 to 12 to the inch, and in precise specifications the number of cuts to the inch must be stated, such as 6-cut, 8-cut, 10-cut, 12-cut. The effect of axeing is to cover the surface with chisel marks which are made parallel as far as practicable. Axeing is a final finish.

Tooth Axed.—The tooth axe is practically a number of points and it leaves the surface of a stone in the same condition as fine pointing. It is usually, however, only a preparation for bush hammering, and the work is then done without regard to effect so long as the surface of the stone is sufficiently leveled.

Bush Hammered.—The roughnesses of a stone are pounded off by the bush hammer, and the stone is then said to be "bushed." This kind of finish is dangerous on sandstone, as experience has proved that sandstone thus treated is very apt to scale.

Fig. 23.



In dressing limestone which is to have a bush hammered finish, the usual sequence of operations is: 1st, rough pointing; 2d, tooth axeing; 3d, bush hammering.

Rubbed.—In dressing sandstone and marble, it is very common to give the stone a plane surface at once by the use of the stone saw. Any roughnesses left by the saw are removed by rubbing with

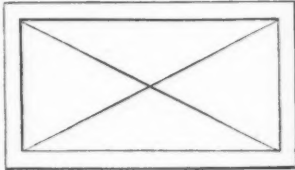
grit or sandstone. Such stones, therefore, have no margins. They are frequently used in architecture for string courses, lintels, door jambs, &c., and they are also well adapted for use in facing the walls of lock chambers, and in other localities where a stone surface is liable to be rubbed by vessels or other moving bodies.

Fig. 24.



Diamond Panels.—Sometimes the space between the margins is sunk immediately adjoining them and then rises gradually until the four planes form an apex at the middle of the panel. Such panels are called diamond panels, and in the case described, the panel is

Fig. 25.



a sunk diamond panel. When the surface of the stone rises gradually from the inner lines of the margins to the middle of the panel, it is called a raised diamond panel. Both kinds of finish are common on bridge quoins and similar work. The details of this method of dressing should be given in the specifications.

MASONRY.

As the term stone masonry includes properly all classes of construction in stone which require the employment of skilled mechanics or masons, any class of masonry may be laid dry, in lime mortar or in cement mortar, at will. On this point specifications should always be precise.

(1) *RUBBLE MASONRY.*—This is composed of unsquared stones. It may be *Uncoursed Rubble* (Fig. 26), laid without any attempt at regular courses, or *Coursed Rubble* (Fig. 27), levelled off at specified heights to a horizontal surface. The stone may be required to be roughly shaped with the hammer, so as to fit approximately.

(2) *SQUARED STONE MASONRY.*—According to the character of the face, this is classified as *Quarry-faced* (Fig. 28), or as *Pitch-faced* (Fig. 29). If laid in regular courses of about the same rise throughout, it is *Range work* (Fig. 30). If laid in courses that are not continuous throughout the length of the wall, it is *Broken Range work* (Fig. 31). If not laid in courses at all, it is *Random work* (Fig. 32), and this is generally to be

expected of this kind of masonry, unless the specifications call for *Range* work.

In quarry-faced and pitch-faced masonry, quoins and the sides of openings are usually *hammer-dressed*. This consists in removing projections so as to secure a rough-smooth surface, and is done with the face hammer, the plain axe, or the tooth axe. This work is a necessity where door or window frames are inserted, and it greatly improves the general effect of the wall if used wherever a corner is turned.

(3) **ASHLAR MASONRY.**—This is equivalent to “cut-stone masonry,” or masonry composed of any of the various kinds of cut-stone mentioned above. As a rule the courses are continuous (Fig. 33), but sometimes they are broken by the introduction of smaller stones of the same kind, and then it is called *Broken Ashlar* (Fig. 34). If the stones are less than one foot in height, the term *Small Ashlar* is proper. The term *Rough Ashlar* is sometimes given to squared stone masonry either “quarry-faced,” or “pitch-faced,” when laid as Range work; but it is believed that it is more logical and more expressive to call such masonry “Squared range work.” From its derivation, Ashlar apparently means large, square blocks, but practice seems to have made it synonymous with “cut stone,” and this secondary meaning has been retained for convenience.

Dimension-stones are cut stones, all of whose dimensions have been fixed in advance. If the specifications for Ashlar masonry are so written as to prescribe the dimensions to be used, it will not be necessary to make a new class of such stones.

Range work, whether of squared stones or of ashlar, is usually backed up with Rubble masonry, which in such cases is specified as coursed Rubble.

Whatever terms are applied in common use, to various classes of masonry, it is not safe to trust to them alone in preparing specifications for construction, but every specification should contain an accurate description of the character and quality of the work desired. Whenever practicable, samples of such kind of cutting and masonry should be prepared beforehand, and exhibited to the persons who propose to undertake the work.

TOOLS
USED IN STONE CUTTING

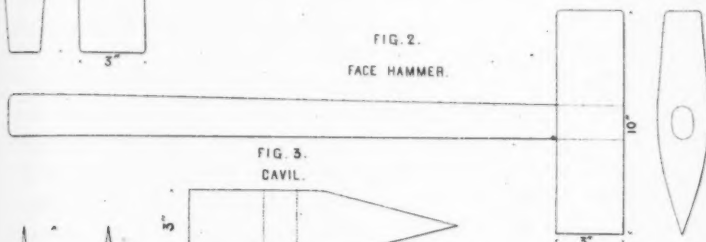
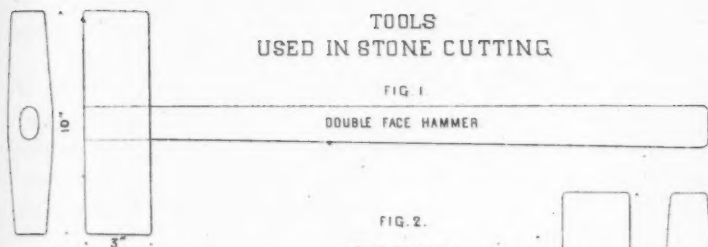


FIG. 3.
CAVIL.

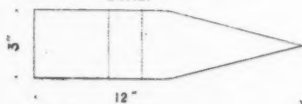


FIG. 4.
PICK.

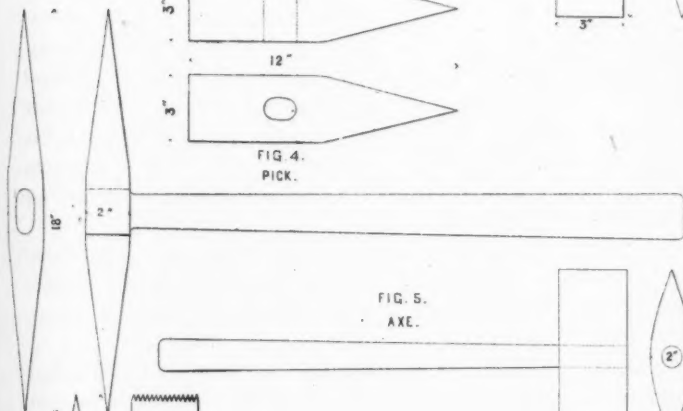


FIG. 6.
TOOTH AXE.

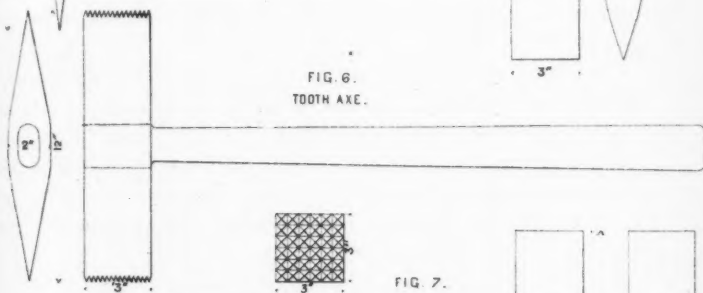
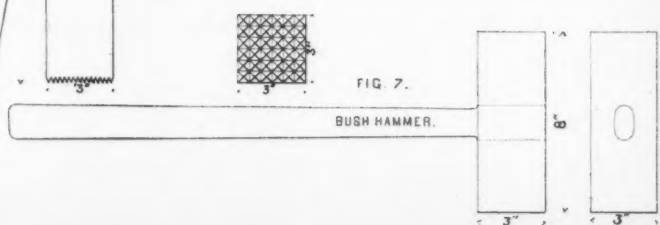
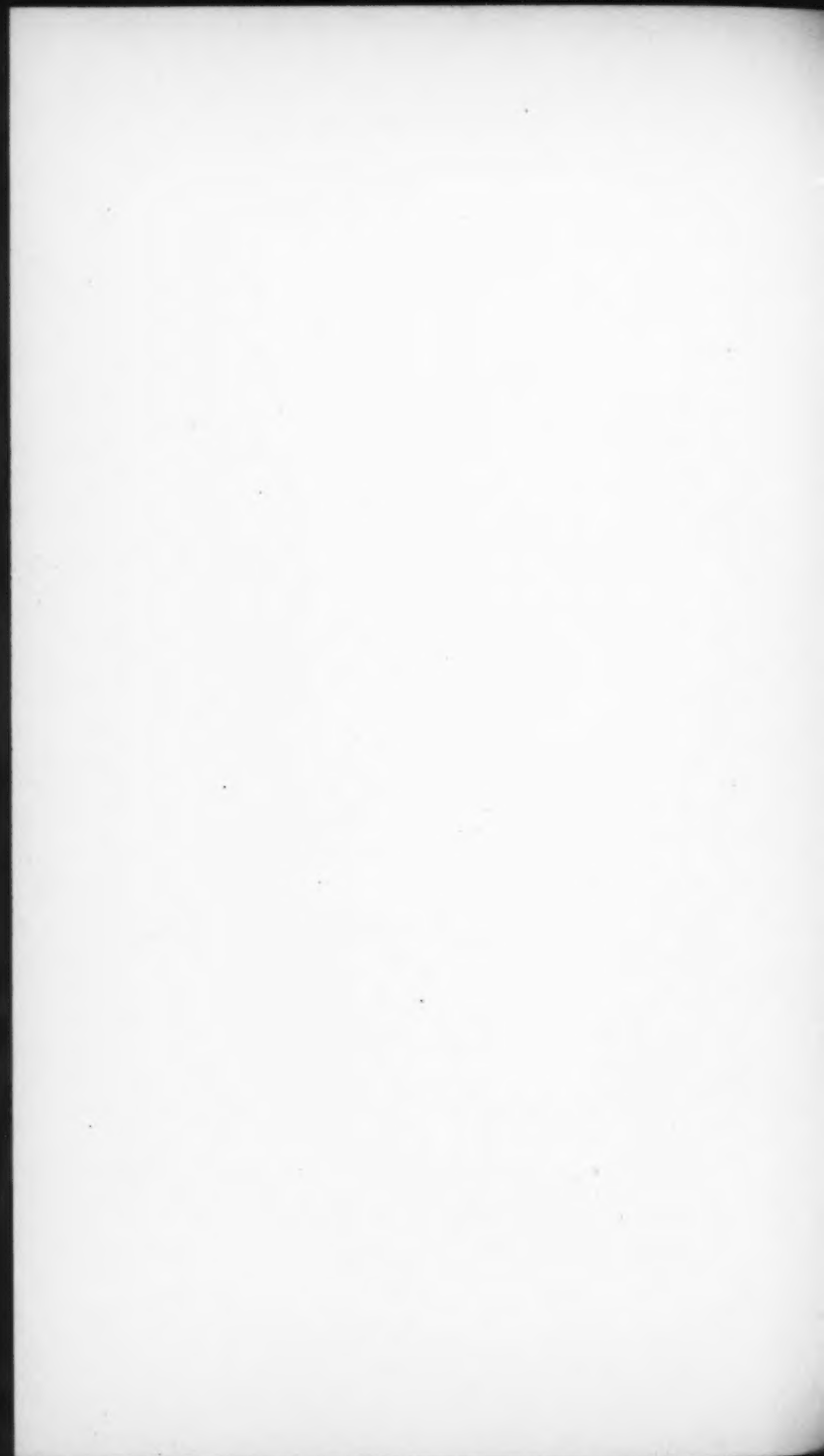
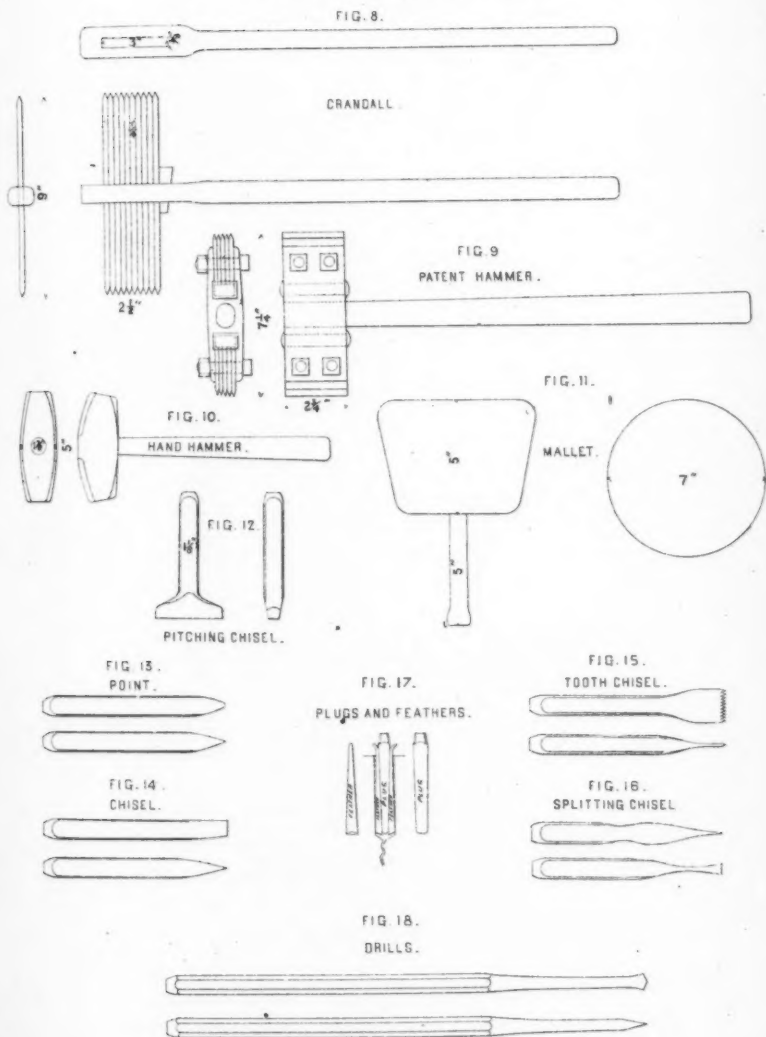


FIG. 7.





TOOLS USED IN STONE CUTTING.





RUBBLE.

Fig. 26
COURSED RUBBLE.

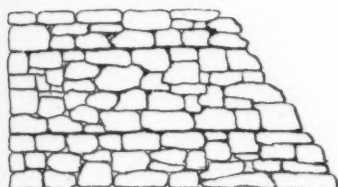
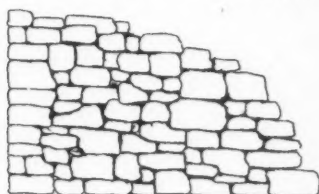


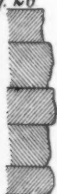
Fig. 27
UNCOURSED RUBBLE.



SQUARED STONE.

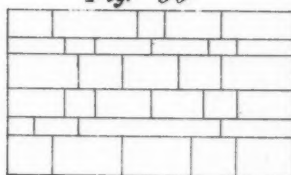
QUARRY PITCH

Fig. 28 FACED *Fig. 29*



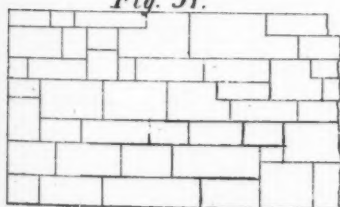
RANGE.

Fig. 30



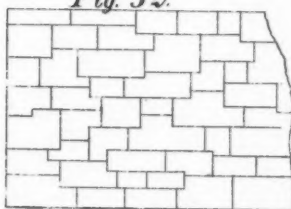
BROKEN RANGE

Fig. 31.



RANDOM.

Fig. 32.



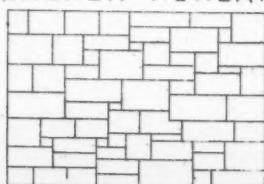
ASHLAR.

Fig. 33 ASHLAR.



Fig. 34

BROKEN ASHLAR.





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DISCUSSION

ON LEVEES AS A SYSTEM FOR RECLAIMING LOW LANDS.*

MR. J. FOSTER FLAGG.—I desire to make a few remarks upon a theory referred to in this paper† and combatted by Prof. Forshey (although I think upon erroneous grounds), in its discussion,‡ viz., the supposed tendency of southward flowing rivers, in the northern hemisphere, to wear their way towards the west.

An exaggerated idea seems to be formed by many of the amount of this tendency; and although willing to admit that there is some inclination in such rivers to wear in that direction, yet I will endeavor to show that its amount is comparatively trifling, and therefore that it may easily

* Referring to Paper No. CXXI, Vol. V, Page 115. † Vol. V, p. 134. ‡ Vol. V, p. 301.

be overcome by local causes. I am aware that a number of rivers, notably the affluents of the Plata, the Euphrates, the Ganges, the Nile, the Rhine, the Elbe, the Vistula, the Volga, etc., are cited as confirming this hypothesis, but it is difficult for any one, without actual familiarity with the topography of the localities of these rivers, to judge whether it is correctly assumed that the change in their course is fairly due to the cause assigned, and not to some others purely local, and not even then without considerable study. No ordinary maps would be sufficient for predicating such an opinion.

If the mere absence of any bluffs on the western bank of a river were sufficient to ensure its working its way over in that direction, I cannot see why the law, if it is a positive law, should not hold good for the Mississippi river, one of such magnitude, and flowing for such a distance of its course nearly due south. Mr. Bayley gives as an explanation, that after the formation of large bends or loops, they are by the continued cutting action of the river broken through and left to the westward. But why of necessity should these loops be always cut through at their eastern extremities; why on the average should not as many be cut through to the westward, and the loops left to the eastward? I cannot see why the Mississippi should *of necessity* make its cut-offs in this way more than any other southern bound rivers with alluvial banks. It may be that from some local cause the loops are more easily cut off and left to the westward; but if so, it only shows the ease with which such slight local causes can overcome the tendency we are discussing.

On the other hand, Professor Forshey states * that "the earth in its rotation revolves as a whole, water and all, and there is no appreciable tendency of water rather than solids to incline to the west." This is true so long as the water, like the solids, *remains fixed in one spot* on the earth's surface, but is no longer so when the water is flowing from one latitude to another. It seems to me that, speaking of the velocity of 900 miles per hour of rotation, he is confounding the *absolute* velocity, due to the rotation of the earth, of a particle of water in the river (which would be about what he states at the latitude of 30°), with its *increased* velocity in an easterly direction due to its movement towards the equator, the resistance to this *increase* of velocity being what is supposed to produce the wearing of the river westward.

* Vol. V, p. 301.

If we could imagine a stream of water flowing due south at the latitude of 35° , to be thenceforward free to flow unconfined by solid banks, and moreover free from the friction or inertia of any surrounding fluid, the stream would in twenty-four hours attain a velocity of sixteen miles per hour due west in addition to its assumed velocity of four miles per hour to the south, and such increase in the resultant velocity and change in its direction would have, naturally, a tremendous cutting action upon any bank opposed to it; but in point of fact the west bank of the river *constantly* resists any such change of direction, or increase of velocity, which, therefore, in a river whose general course is south, would in the aggregate amount to nothing.

In order to investigate more closely this action, let us take for an example a river similar in size to the Mississippi—say with a width of 4 000 feet, sectional area of 190 000 square feet, and fall of 0.4 feet per mile. By Humphrey & Abbott's formula, the velocity then would be 6.6 feet per second = 4.5 miles per hour, and the discharge 1 254 000 cubic feet per second.

I.—First let us assume that the river is running due south with straight banks free from any bends or curves, and see what will be the effect at the latitude of 35° (farther south the increase in velocity of rotation is less rapid, and any effect resulting therefrom necessarily less). Let us also assume that for a short distance the surface of the earth is conical; the length of a degree of longitude, going south, would then increase uniformly, and the *acceleration* of the velocity of rotation of a particle of water, flowing south with a constant motion, would be *constant*. In one hour the increase of velocity of rotation, or velocity in an easterly direction, would be 0.9914 feet per second, and consequently the acceleration per second $p = \frac{0.9914}{3600} = 0.000\ 275$ feet.

The force to produce this acceleration, calling the mass of the particle M , would be Mp , and this force is simply produced by a constant, steady pressure from the west bank. A square foot of vertical surface on the west bank would have to exert this pressure for the whole width of the river, and the total amount would be

$$\frac{4000 \times 62.3}{32.1} \times 0.000\ 275 = 2.135 \text{ lbs.}$$

only to the square foot, equivalent to a head of water of 0.41 inches—a trifling amount. As the friction of water is held to be independent of

pressure, it is hard to see how this slight change of pressure to the west bank, with no increase in the velocity of current or change in its direction, can increase the scour upon that bank.

It may also be considered in this way : as each unit in mass of water is drawn downwards by the force 32.1, and westwards by the force 0.000 275, the surface of water in the river takes a position normal to the resultant of these two forces ; from similar triangles we form the proportion

$$32.1 : 0.000\ 275 :: 4000 : x$$

which gives us for the difference in level of the west shore over the east shore, due to this heaping up of the water, 0.034 feet or 0.41 inches, as before.

II.—Next let us consider a section of the river with a straight course, and running west of south. Here the inertia of the particle of water resisting its increase of velocity to the east, will be divided into two components, one normal to the bank, and taken up by the bank as before, and the other parallel to the current increasing its velocity. This effect upon the velocity of the current will be a maximum when the river is running due southwest. The acceleration in direction of current will then become 0,000 1375 feet per second.* The acceleration of gravity which keeps up the velocity of 4.5 miles per hour is

$$32.1 \frac{0.4}{5280} = 0.002\ 439\ 6$$

adding the above acceleration 0.000 137 5

we have the total 0.002 577 1

which is the total acceleration to produce velocity, or total force for a mass = 1, equivalent in effect to a fall of 0.424 feet per mile in place of 0.4 feet ; and the corresponding velocity would be 6.69 feet per second, an increase of about $\frac{1}{4}$.

* Let α = angle of river with meridian.

v = velocity of current,

v_1 = its velocity south,

then, $v_1 = v \cos \alpha$

Putting p = acceleration when flowing due south,

p_1 = " " " at angle α

p_2 = " " produced in direction of current.

As p is a direct function of v

$p_1 = p \cos. \alpha$, and $p_2 = p_1 \sin. \alpha = p \sin. \alpha \cos. \alpha$.

p_2 is a maximum when $\alpha = 45^\circ$ when we have

$p_2 = p \sin.^2 45^\circ = \frac{1}{2} p$.

This increased velocity would cause increased scour, but it would not, neglecting extraneous causes, be different on the west from the east bank, and the increased pressure upon the right bank would be less than with the course due south.

With a normal fall in the surface of the water of only 0.15 feet per mile, the relative increase of velocity when flowing southwest would be greater, *i. e.*, the velocity due to fall in river would be 5.14 feet per second, and including that due to rotation of earth, 5.33 feet per second.

III.—Next, let us suppose, as an extreme case, that the river runs due *S. W.* for several miles ; then making an easy turn, runs due *S. E.* for a similar distance, and then again due *S. W.* and so on. Let us also suppose the fall of 0.4 feet to the mile to be uniform throughout, and, in order to get the maximum effect, let us suppose that the section of the river, while flowing *S. E.*, is sufficiently enlarged to make up for the diminution of velocity in this direction. Under these suppositions, the river, whilst flowing *S. W.*, would, after obtaining its regimen, have a velocity of 6.69 feet per second, and whilst flowing *S. E.*, a velocity of 6.51 feet per second. The result would be that the western bends would cut away a little more rapidly, everything being equal, than the eastern ; and as many cut-offs being formed to the west as to the east, the movement of the whole river would be gradually westward.

But in reality there would be no such uniformity of fall, the actual fall of surface going *S. W.* would be diminished, and going *S. E.* increased, keeping the river sections more nearly alike, and the velocities more nearly alike, so that the cutting at western bends would be more nearly equal in amount to that at the eastern than indicated in the previous paragraph.

A difference so slight as this in the cutting action might easily be overcome by local circumstances ; as, for example, the river section might be so enlarged near the western bend as to make the velocity actually less there than at the eastern one, notwithstanding the assistance of the earth's rotation, especially if the succeeding courses run more nearly east and west ; or the clay might be slightly tougher to the west, and so the cutting in that direction be less effective.

IV.—If, again, as at Vicksburg, the river, after flowing *N. E.*, turns and flows *S. W.*, the increase in velocity of current due to rotation of earth, will be the same for each course, and therefore, the cutting at one bend from this cause be no greater than at the next one.

To sum up, then, the conditions must be favorable for any material movement of the channel to the westward ; local circumstances, independent of any cliffs or rock formation being sufficient to easily counteract any such tendency.

ERRATA.

Transactions, Vol. VI, May to August, 1877, inclusive. The numbers of the papers from 140 to 146 inclusive, are wrongly printed—CLX, CLXI, CLXII, CLXIII, CLXIV, CLXV, CLXVI.—Change these to CXL, CXLI, CXLII, CXLIII, CXLIV, CXLV, CXLVI.